N00236.001714
ALAMEDA POINT
SSIC NO. 5090.3

Southwest Division
Naval Facilities Engineering Command
Contracts Department
1220 Pacific Highway, Building 127, Room 112
San Diego, CA 92132-5190

CONTRACT NO. N68711-98-D-5713
CTO NO. 0054

FINAL

FEASIBILITY STUDY REPORT

Revision 0
December 5, 2003

OPERABLE UNIT-3, INSTALLATION RESTORATION SITE 1, 1943-1956 DISPOSAL AREA, ALAMEDA POINT ALAMEDA, CALIFORNIA

VOLUME 2 GEOTECHNICAL AND SEISMIC

DCN: FWSD-RAC-03-3603

Prepared by:

FOSTER WHEELER ENVIRONMENTAL CORPORATION

1230 Columbia Street, Suite 500
San Diego, CA 92101

REGISTERED PROFESSIONAL ENGINEER
ABID R. LOAN
A. L.
No. C 055826
Exp 12/31/04
CIVIL
STATE OF CALIFORNIA

Abid Loan, P.E.
Project Manager

DEPARTMENT OF THE NAVY
SOUTHWEST DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
1220 PACIFIC HIGHWAY
SAN DIEGO, CA 92132-5190

5090
Ser 06CA.CD\1491
November 24, 2003

Ms. Anna-Marie Cook
US EPA
Region IX
75 Hawthorne Street
San Francisco, CA 94105-3901

Dear Ms. Cook:

This letter transmits the Final Feasibility Study (FS) Report for Operable Unit 3 Installation Restoration Site 1, 1943-1956 Disposal Area Alameda Point, Alameda, California Volume 2 Geotechnical and Seismic.

The Draft Final was submitted to your office on September 26, 2003 for review and comment by the United States Environmental Protection Agency. The Navy considers this document to be a primary document of the Federal Facilities Agreement (FFA). As no comments were received 30 days of the Draft Final issuance, the Navy concludes this is an indication of your acceptance of the FS (Volume 2 Geotechnical and Seismic) Report, and is finalizing the document in accordance with the FFA.

Provided for your convenience is the Final Report cover and title page that replaces the Draft Final cover and title page. Please disregard draft final footnotes, as a new copy of the report will not be generated.

If you have any questions, please call Ms. Claudia Domingo, Remedial Project Manager at (619) 532-0935.

Sincerely,

THOMAS L. MACCHIARELLA
BRAC Environmental Coordinator
By direction of the Commander

Encl: (1) Final Feasibility Study Report for Operable Unit 3 Installation Restoration Site 1, 1943-1956 Disposal Area Alameda Point, Alameda, California Volume 2 Geotechnical and Seismic report cover and title page

5090
Ser 06CA.CD\1491
November 24, 2003

Copy to :
Ms. Marcia Liao
Department of Toxic Substances Control
700 Heinz Avenue, Suite 200
Berkeley, CA 94710-2721

Ms. Judy Huang
San Francisco Bay Regional Water Quality Control Board
1515 Clay Street, Suite 1400
Oakland, CA 94612

Ms. Karla Brasaemle (EPA Consultant)
Tech Law, Inc.
530 Howard Street, Suite 400
San Francisco, CA 94105

Mr. Peter Russell, PhD (ARRA Environmental Consultant)
Northgate Environmental Management
950 Northgate Drive, Suite 313
San Rafael, CA 94903

TETRA TECH FW, INC.

TRANSMITTAL/DELIVERABLE RECEIPT

Contract No. N68711-98-D-5713 (RAC III)

Document Control No. 03-3603
File Code: 5.0

TO: Contracting Officer
Naval Facilities Engineering Command
Southwest Division
Ms. Beatrice Appling, 02R1.BA
1220 Pacific Highway
San Diego, CA 92132-5190

DATE: 12/05/03
CTO: 0040
LOCATION: Alameda Point

FROM: Edwin Neil Hart
Neil Hart, Program Manager

DESCRIPTION: Final Feasiblity Study Report, Rev. 0, 12/05/03

TYPE: ☐ Contract/Deliverable ☒ CTO Deliverable ☐ Notification
☐ Other

VERSION: Final
(e.g. Draft, Draft Final, Final, etc.)

REVISION #: 0

ADMIN RECORD: Yes ☒ No ☐ Category ☐ Confidential ☐
(PM to Identify)

SCHEDULED DELIVERY DATE: 10/29/03 ACTUAL DELIVERY DATE: 12/05/03

NUMBER OF COPIES SUBMITTED: O/5C/6E Copy of SAP to N. Ancog ☐

COPIES TO: (Include Name, Navy Mail Code, and Number of Copies)

NAVY:	TtFWI:	OTHER: (Distributed by TtFWI)
C. Domingo(06CA.CD)	A. Eloskof	See Attached Cover Letter for
O/2E	M. Schneider	Additional Distribution
T. Macchiarella(06CM.TM)		
1C/1E		
D. Silva(05GDS)3C/3E		
Basic Contract File(02R1)		Date/Time Received
1C		

Rev. 12/4/03

FOSTER WHEELER

FOSTER WHEELER ENVIRONMENTAL CORPORATION

TRANSMITTAL/DELIVERABLE RECEIPT

Contract No. N68711-98-D-5713 (RAC III)

Document Control No. 03-1368

File Code: 5.0

TO: Contracting Officer
Naval Facilities Engineering Command
Southwest Division
Ms. Beatrice Appling, 02R1.BA
1220 Pacific Highway
San Diego, CA 92132-5190

DATE: 09/29/03
CTO: 0054
LOCATION: Alameda Point

FROM:

Neil Hart, Program Manager

DESCRIPTION: Draft Final Feasibility Study Report, Rev. 0, 09/26/03

TYPE: ☐ Contract/Deliverable ☒ CTO Deliverable ☐ Notification ☐ Other

VERSION: Draft Final (c.g. Draft, Draft Final, Final, etc.)

REVISION #: 0

ADMIN RECORD: Yes ☒ No ☐ Category ☐ Confidential ☐
(PM to Identify)

SCHEDULED DELIVERY DATE: 07/18/03 ACTUAL DELIVERY DATE: 09/29/03

NUMBER OF COPIES SUBMITTED: O/5C/6E

COPIES TO: (Include Name, Navy Mail Code, and Number of Copies)

NAVY:	FWENC:	OTHER: (Distributed by FWENC)
R. Weissenborn (06CARW) O/2E	M. Schneider	See Attached Cover Letter for Additional Distribution
D. Silva (056DS) 3C/3E	A. Loan	
M. McClelland (06CAMM) 1C/1E	J. Dessort	
Basic Contract File (02R1) 1C		Date/Time Received

DEPARTMENT OF THE NAVY
SOUTHWEST DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
1220 PACIFIC HIGHWAY
SAN DIEGO, CA 92132-5190

5090
Ser 06CA.RW\1326
September 25, 2003

Mr. Mark Ripperda
US EPA
Region IX
75 Hawthorne Street
San Francisco, CA 94105-3901

Dear Mr. Ripperda:

This letter transmits the *Draft Final Feasibility Study Report, Operable Unit-3, Site 1, 1943-1956 Disposal Area, Alameda Point, Alameda, California Volume 2 Geotechnical and Seismic* for your review and comment. The document is being submitted far in advance of Volume 1 to expedite closeout of the contract and to free the contractor's staff for other work. In accord with the Federal Facility Agreement, this document will become final in 30 days.

Please feel free to contact me at (619) 532-0952, if you have any questions.

Sincerely,

Richard C. Weissenborn

RICHARD C. WEISSENBORN, P.E.
Remedial Project Manager

Encl: (1) *Draft Final Feasibility Study Report, Operable Unit-3, Site 1, 1943-1956 Disposal Area, Alameda Point, Alameda, California Volume 2 Geotechnical and Seismic*

Copy to:
Ms. Marcia Liao
Department of Toxic Substances Control
700 Heinz Avenue, Suite 200
Berkeley, CA 94710-2721

Ms. Judy Huang
San Francisco Bay Regional Water Quality Control Board
1515 Clay Street, Suite 1400
Oakland, CA 94612

FINAL FEASIBILITY STUDY REPORT FOR OPERABLE UNIT 3, INSTALLATION RESTORATION SITE 1, 1943-1956 DISPOSAL AREA

VOLUME 1 IS NOT ON FILE

EXTENSIVE RESEARCH WAS PERFORMED BY NAVFAC SOUTHWEST TO LOCATE THE ORIGINAL VERSION. THIS PAGE HAS BEEN INSERTED AS A PLACEHOLDER AND WILL BE REPLACED SHOULD THE MISSING ITEM BE LOCATED.

QUESTIONS MAY BE DIRECTED TO:

DIANE C. SILVA
RECORDS MANAGEMENT SPECIALIST
NAVAL FACILITIES ENGINEERING COMMAND SOUTHWEST
1220 PACIFIC HIGHWAY
SAN DIEGO, CA 92132

TELEPHONE: (619) 532-3676

TABLE OF CONTENTS

PAGE

TABLE OF CONTENTS
(Continued)

PAGE

TABLE OF CONTENTS
(Continued)

LIST OF TABLES

PAGE

LIST OF FIGURES

LIST OF FIGURES
(Continued)

FOLLOWING PAGE

LIST OF FIGURES
(Continued)

LIST OF FIGURES
(Continued)

FOLLOWING PAGE

ABBREVIATIONS AND ACRONYMS

ARAR	applicable or relevant and appropriate requirement
bpf	blows per foot
bgs	below ground surface
BRAC	Base Realignment and Closure
CCR	California Code of Regulations
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
CFR	Code of Federal Regulations
CPT	cone penetrometer test
CTO	Contract Task Order
CWA	Clean Water Act
DERP	Defense Environmental Restoration Program
DO	Delivery Order
EFANW	Engineering Field Activities Northwest
EPA	U.S. Environmental Protection Agency
FS	Feasibility Study
FWENC	Foster Wheeler Environmental Corporation
g	acceleration due to gravity
H	height of sheet pile
HAI	Hushmand Associates, Inc.
IR	Installation Restoration
kips/foot	kips per linear foot
MCE	maximum credible earthquake
MPE	maximum probable earthquake
msl	mean sea level
N/A	not applicable
NAS	Naval Air Station
O&M	operation and maintenance
OMB	Office of Management and Budget
OU	Operable Unit
pcf	pounds per cubic feet

ABBREVIATIONS AND ACRONYMS
(Continued)

PHGA	peak horizontal ground acceleration
PP	Proposed Plan
psf	pounds per square foot
psi	pounds per square inch
RAC	Remedial Action Contract
RCRA	Resource Conservation and Recovery Act
RI	Remedial Investigation
ROD	Record of Decision
RWQCB	Regional Water Quality Control Board
SHANSEP	Stress History and Normalized Engineering Properties
SOW	Scope of Work
SPT	standard penetration test
SWDIV	Southwest Division Naval Facilities Engineering Command
TtEMI	Tetra Tech EM, Inc.
USC	United States Code

EXECUTIVE SUMMARY

This Draft Final Geotechnical Feasibility Study (FS) Report provides a recommended remedial alternative for addressing the geotechnical and seismic hazards identified in the Remedial Investigation (RI) Report Addendum, Volume III, at Installation Restoration (IR) Site 1 in Operable Unit (OU)-3 of the former Naval Air Station (NAS) Alameda, Alameda Point, Alameda, California. The scope of work for this FS includes outlining the remedial action objective, identifying response actions, developing and screening remedial alternatives, detailing implementability analysis, cost evaluation of selected remedial alternatives, and selecting a remedial alternative. U.S. Environmental Protection Agency (EPA) guidelines governing FS preparation for Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) sites were followed.

A separate Environmental FS Report is currently being prepared to address groundwater contamination at the site [Tetra Tech EM, Inc. (TtEMI), 2002a]. Preliminary options for remediation include a funnel and gate reactive wall in a groundwater plume area near the shoreline. This environmental remedial alternative may impact the selected geotechnical remedial alternative. However, at this time, the level of contamination is still being investigated. Therefore, the geotechnical remedial alternatives considered did not directly take into account any potential impact of the environmental alternatives. During the detailed design stage, design efforts for both remedial actions will be coordinated to ensure that actions taken to address geotechnical and seismic hazards do not negatively impact the remedial action for the groundwater plume, and vice versa.

The City of Alameda has proposed that IR Site 1 be used as a golf course after transfer from the Navy. Geotechnical and seismic hazards, identified in the *Final Ordnance and Explosives Waste Characterization Report* [Foster Wheeler Environmental Corporation (FWENC), 2002], also referred to as the RI Report Addendum, Volume III, in this document, include static and seismic slope instability and liquefaction potential. These hazards could lead to liquefaction-induced settlements and relatively large permanent lateral deformations. Due to the former use of the site as a landfill and its proximity to San Francisco Bay, the main concern is release of waste into San Francisco Bay as a result of slope instability and liquefaction-induced lateral spreading. The magnitude of permanent lateral deformations due to the site design earthquake [maximum credible earthquake (MCE), defined as the largest earthquake that can reasonably be expected to occur based on known geologic and seismologic data (Day, 2002)] was estimated to be up to 19 feet (FWENC, 2002). In addition, liquefaction-induced lateral spreading was estimated to be greater than 20 feet and much higher in some areas (up to 260 feet) (FWENC, 2002).

Based on these geotechnical and seismic hazards (FWENC, 2002), it was determined that the remedial action objective would be to prevent release of waste into San Francisco Bay. This can be accomplished by improving slope stability and/or reducing potential lateral deformations. Technical performance criteria were established to determine if the remedial alternatives selected to mitigate the identified geotechnical and seismic hazards could satisfy the remedial action objective. The performance criteria were developed for both static and seismic loading conditions. For static loads, a slope stability factor of safety of at least 1.5 for various slopes across the site is required. This factor is defined as the ratio of resisting (stabilizing) forces to the driving forces trying to displace the slope. It is derived from the requirements by the state of California.

Seismic stability evaluation is based on estimating seismically induced slope deformations and the post-earthquake static factor of safety. For seismic events, the pseudo-static slope stability factor of safety should be greater than 1.0. This requires the slope to resist seismic loads and not yield (move). The pseudo-static slope stability factor of safety required is less than the static factor of safety requirement since the pseudo-static slope stability factor of safety also takes into account additional loading caused by the predicted peak horizontal ground acceleration (PHGA) at the site. The PHGA is the largest (absolute) value of horizontal acceleration recorded or expected at the site. For steep slopes, poor soil conditions, or high PHGA, the pseudo-static slope stability factor of safety calculated is usually less than one, and the slopes are expected to yield. Based on the construction history and aerial photographs showing locations of disposal areas, it is estimated that the width of the buffer zone between the waste limit and the shoreline along San Francisco Bay or the Oakland Inner Harbor Channel is approximately 8 to 15 feet wide. In order to satisfy the remedial action objective of preventing waste release into San Francisco Bay, the allowable lateral displacement should be less than the minimum width of the buffer zone (8 feet). Since direct measurements of the width of the buffer zone are not available, an allowable lateral displacement of 4 feet was selected to provide an adequate safety margin (safety factor of 2) if slopes yield during a seismic event at IR Site 1. For post-earthquake stability, a static factor of safety greater than 1.0 is required. This factor is calculated using post-earthquake (residual) strength parameters.

Possible remedial methods available to mitigate the geotechnical and seismic hazards can be classified under two types of general approaches or response actions. The response actions identified included performing soil improvement and/or installing physical buttresses along the shoreline perimeter of the site. Different types of soil improvement methods were evaluated. The improvement methods considered for this FS included: 1) installation of wick drains, 2) application of surcharge (additional fill placement for each consolidation), 3) installation of stone columns, 4) installation of a soil cement gravity wall, 5) excavation along shoreline and soil backfill, and 6) partial solidification. The types of physical buttresses considered for this FS included: 1) drilled concrete caissons, 2) sheet piles, 3) soil bentonite cutoff wall, 4) riprap embankment with soil backfill, 5) inclined timber piles, 6) vibrated beam cement bentonite cutoff wall, 7) vibrated beam Impermix cutoff wall, 8) concrete wall, 9) pre-cast concrete piles, and 10) excavation with riprap.

Since some of the soil improvement and physical buttress-type methods individually may not satisfy the established performance criteria, a combination of remedial methods were developed as remedial alternatives to achieve this objective. The remedial alternatives were combined based on their performance in similar applications and cost effectiveness. A total of 20 remedial alternatives were considered. These included: 1) wick drains with surcharge, 2) stone columns with surcharge, 3) sheet piles with anchors, 4) stone columns with surcharge and sheet piles, 5) soil cement gravity wall and stone columns, 6) concrete wall, 7) excavation and backfill with riprap, 8) drilled concrete piers, 9) pre-cast concrete piles, 10) wick drains with surcharge and sheet piles, 11) excavation along shoreline and soil backfill, 12) partial in situ solidification, 13) soil bentonite cutoff wall, 14) riprap embankment in the bay and soil backfill, 15) inclined timber piles, 16) consolidation with surcharge, 17) wick drains with a vacuum, 18) vibrated beam cement bentonite cutoff wall, 19) vibrated beam Impermix cutoff wall, and 20) soil cement gravity wall.

Each alternative was evaluated using EPA criteria for CERCLA sites. Nine evaluation criteria were specified, which include: 1) overall protection of human health, 2) compliance with applicable or relevant and appropriate requirements (ARARs), 3) long-term effectiveness and permanence, 4) reduction of toxicity, mobility, and volume through treatment, 5) short-term effectiveness, 6) implementability, 7) cost, 8) state or support agency acceptance, and 9) community acceptance. An initial screening evaluation was performed to reduce the number of remedial alternatives before detailed analyses were performed. The screening evaluation was based on the following EPA screening factors, which included effectiveness, implementability, and cost. The effectiveness evaluation was associated with the first five of the nine evaluation criteria. After the initial screening process, nine alternatives (Alternatives 1-9) were selected for detailed analyses. The alternatives that were not selected were not evaluated further (Alternatives 10-20).

The nine remedial alternatives selected were analyzed for implementability. Alternatives 1, 3, and 7 were considered not technically feasible, reducing the number of potential remedial alternatives to six. Based on the cost analysis, two of the six alternatives (Alternative 2 and 4) were considered cost-prohibitive compared to the other four (Alternatives 5, 6, 8, and 9).

A final comparative analysis using the nine EPA evaluation criteria was performed on the remaining four alternatives. Based on the comparative analysis, Alternative 5, soil cement gravity wall and stone columns, was determined to be the most feasible. This alternative was selected because of the overall safety and reliability of the soil cement gravity wall compared to the methods proposed in the other three alternatives. Other criteria considered, such as compliance with ARARs, reduction of toxicity, mobility, and volume through treatment, and cost, did not have a significant impact in the screening process because of similar performance related to each of these evaluations criterion. The alternatives evaluated are determined to be necessary for improving site stability. A contaminant-specific FS is underway separately to address the risk posed by chemicals at the site.

Alternative 5 involves the construction of a 24-foot-wide soil cement gravity wall in the Young Bay Mud layer with a thickness varying from 15 to 35 feet along the shoreline perimeter of the site. It also includes the installation of stone columns in the fill layer (from the ground surface to the top of the Young Bay Mud layer) to reduce liquefaction potential by consolidating the liquefiable fill material.

The engineering analysis of this alternative indicated that calculated long-term pre- and post-earthquake slope stability static factors of safety were 3.03 and 2.13, respectively. The estimated lateral deformation was 1.9 feet, considerably less than the 4-foot limit established in the performance criteria. Total cost of this alternative is estimated to be $13,814,190, which is at the lower end of the cost range for all alternatives considered in the cost analysis.

It is recommended that during the detailed design stage, the extent of the remedial measure (area and depth of application) should be further evaluated to determine if these can be reduced (optimized) based on the following:

- Detailed waste delineation along shoreline perimeter
- More sophisticated detailed analysis (such as Finite Element modeling) to obtain more accurate assessment of slope movement
- Risk assessment to determine impact of waste release into the San Francisco Bay/Oakland Inner Harbor Channel

Also, if additional information gathered or evaluations performed during the detailed design stage demonstrate viability of other remedial actions (components) over the preferred alternative (Alternative 5), then the preferred remedy may be altered. For example, the stone cement walls could be extended from the ground surface through the bottom of the Young Bay Mud layer, replacing the stone columns in the upper fill layer. It must be emphasized that any changes made to the preferred remedial alternative must undergo the same screening process to evaluate technical implementability, cost, and long-term effectiveness. Any changes will also need to be re-approved once the preferred remedial alternative has been approved and documented in the final Record of Decision for OU-3, following issuance of the Proposed Plan and consideration of public comments.

1.0 INTRODUCTION

The Southwest Division Naval Facilities Engineering Command (SWDIV) authorized Foster Wheeler Environmental Corporation (FWENC) to prepare a Geotechnical Feasibility Study (FS) Report which provides a recommended remedial alternative for addressing the geotechnical and seismic hazards identified in the Remedial Investigation (RI) Report Addendum, Volume III, of Installation Restoration (IR) Site 1, Operable Unit (OU)-3 of former Naval Air Station (NAS) Alameda, Alameda Point, Alameda, California (Figure 1-1).

The authorization for this work was originally issued under Engineering Field Activities Northwest (EFANW) Remedial Action Contract (RAC) II No. N44255-95-D-6030, Delivery Order (DO) No. 0095, under the Defense Environmental Restoration Program (DERP) for Base Realignment and Closure (BRAC). The performance period under RAC II No. N44255-95-D-6030 expired on September 30, 2002, the close of the federal fiscal accounting period. A new Contract Task Order (CTO) No. 0054, describing the current geotechnical FS under a revised Scope of Work (SOW), was issued under SWDIV RAC III No. N68711-98-D-5713. The new CTO authorizes FWENC to complete all remaining work originally authorized under DO No. 0095.

The work performed in this report is a component of the Navy's RI/FS of the site under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), more widely known as "Superfund." The Navy and regulatory agencies have previously agreed to prepare separate reports for the RI and the FS. The RI Report was prepared by Tetra Tech EM, Inc. (TtEMI, 1999). Additional work performed was reported in three RI Report Addendums: 1) RI Addendum, Volume I – Data Gap Summary Report (TtEMI, 2001), 2) RI Addendum, Volume II – Cumulative Human Health Risk Assessment (TtEMI, 2002b), and 3) RI Addendum, Volume III – *Final Ordnance and Explosives Waste/Geotechnical Characterization Report* (FWENC, 2002).

A separate Environmental FS Report is currently being prepared to address groundwater contamination at the site (TtEMI, 2002a). Preliminary options for remediation include a funnel and gate reactive wall in a groundwater plume area near the shoreline. This environmental remedial alternative may impact the selected geotechnical remedial alternative. However, at this time, the level of contamination is still being investigated. Therefore, the geotechnical remedial alternatives considered in the Geotechnical FS Report (included herein) do not directly take into account any potential impact of the environmental alternatives. During the detailed design stage, design efforts for both remedial actions will be coordinated to ensure that actions taken to address geotechnical and seismic hazards do not negatively impact the remedial action for the groundwater plume and vice versa.

This Geotechnical FS Report will primarily use data from the *Final Ordnance and Explosives Waste/Geotechnical Characterization Report* (RI Addendum, Volume III) (FWENC, 2002). In its entirety the IR Site 1 FS Report will consist of TtEMI's Environmental FS Report, Volume 1, and the Geotechnical FS Report, annotated as Volume 2.

The purpose of this CTO is to perform a FS of remedial alternatives to mitigate geotechnical and seismic hazards identified in the *Finalt Ordnance and Explosives Waste/Geotechnical Characterization Report* (referred to as the RI Report Addendum, Volume III in this FS) (FWENC, 2002). This Geotechnical FS Report is limited to a feasibility evaluation of the proposed remedial alternatives and provides a recommended alternative to address these hazards.

This Geotechnical FS Report is organized as follows:

- **Section 1.0, Introduction** – Section 1.0 presents the site background, including its history and geology, and reviews the geotechnical and seismic hazards associated with the site.
- **Section 2.0, Development of Remedial Action Objective, Response Actions, and Performance Criteria** – Section 2.0 establishes specific technical performance criteria that each remedial alternative must satisfy. General response actions to mitigate identified hazards are proposed, including a list of specific remedial alternatives. A preliminary evaluation was performed to evaluate the effectiveness of the general response actions.
- **Section 3.0, Development and Screening of Alternatives** – Section 3.0 describes the development of 20 remedial alternatives. These alternatives are then screened based on U.S. Environmental Protection Agency (EPA) evaluation criteria for CERCLA sites. After the screening process, remaining alternatives are subject to more detailed analysis.
- **Section 4.0, Detailed Analysis of Selected Alternatives** – Section 4.0 provides a detailed description and implementability analysis of the remaining remedial alternatives. Based on the analysis, three of the remedial alternatives are eliminated from consideration. A final comparative analysis using nine CERCLA evaluation criteria was performed to evaluate the remaining alternatives and to identify a recommended alternative for implementation.

FWENC's seismic/geotechnical subconsultant, Hushmand Associates, Inc. (HAI), provided input to Sections 2.0 and 3.0 and performed detailed technical analysis of the nine selected remedial alternatives in Section 4.0 (Attachment 1).

A summary of the FS process is detailed in a flowchart presented in Figure 1-2.

1.1 BACKGROUND INFORMATION

IR Site 1 is located at the northwestern corner of Alameda Point, Alameda, California (see Figure 1-1). The site makes up OU-3 of former NAS Alameda. Alameda Point is located on the

DRAWING NO: 03136811.DWG

DCN: FWSD-RAC-03-1368

CTO: # 0054

APPROVED BY: AL

CHECKED BY: TL

REVISION: 0

DRAWN BY: MD

DATE: 09/26/03

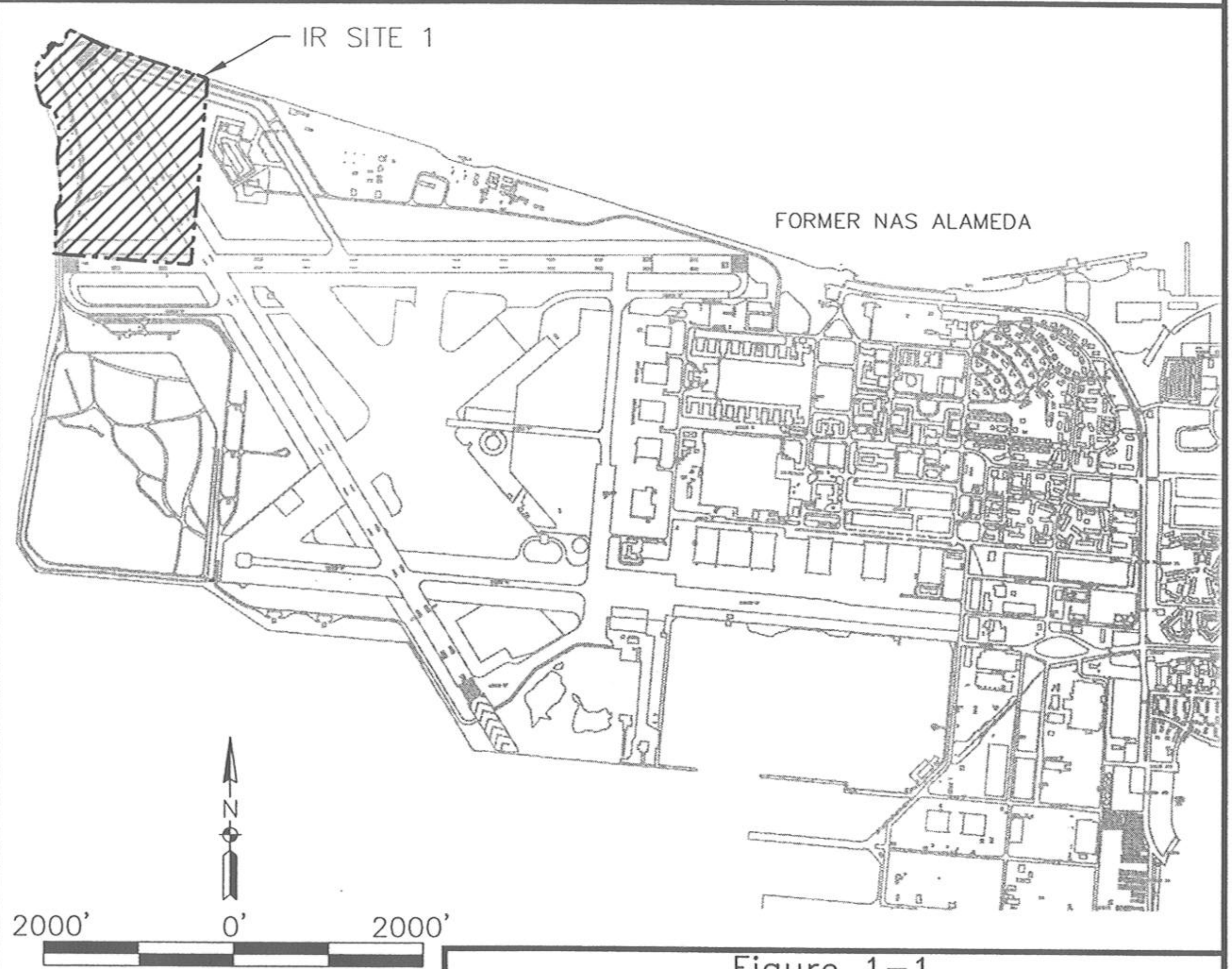


Figure 1-1
ALAMEDA POINT VICINITY MAP
ALAMEDA, CA

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\CTO-0054\DWG\031368\03136811.DWG
PLOT/UPDATE: SEP 12 2003 15:31:40

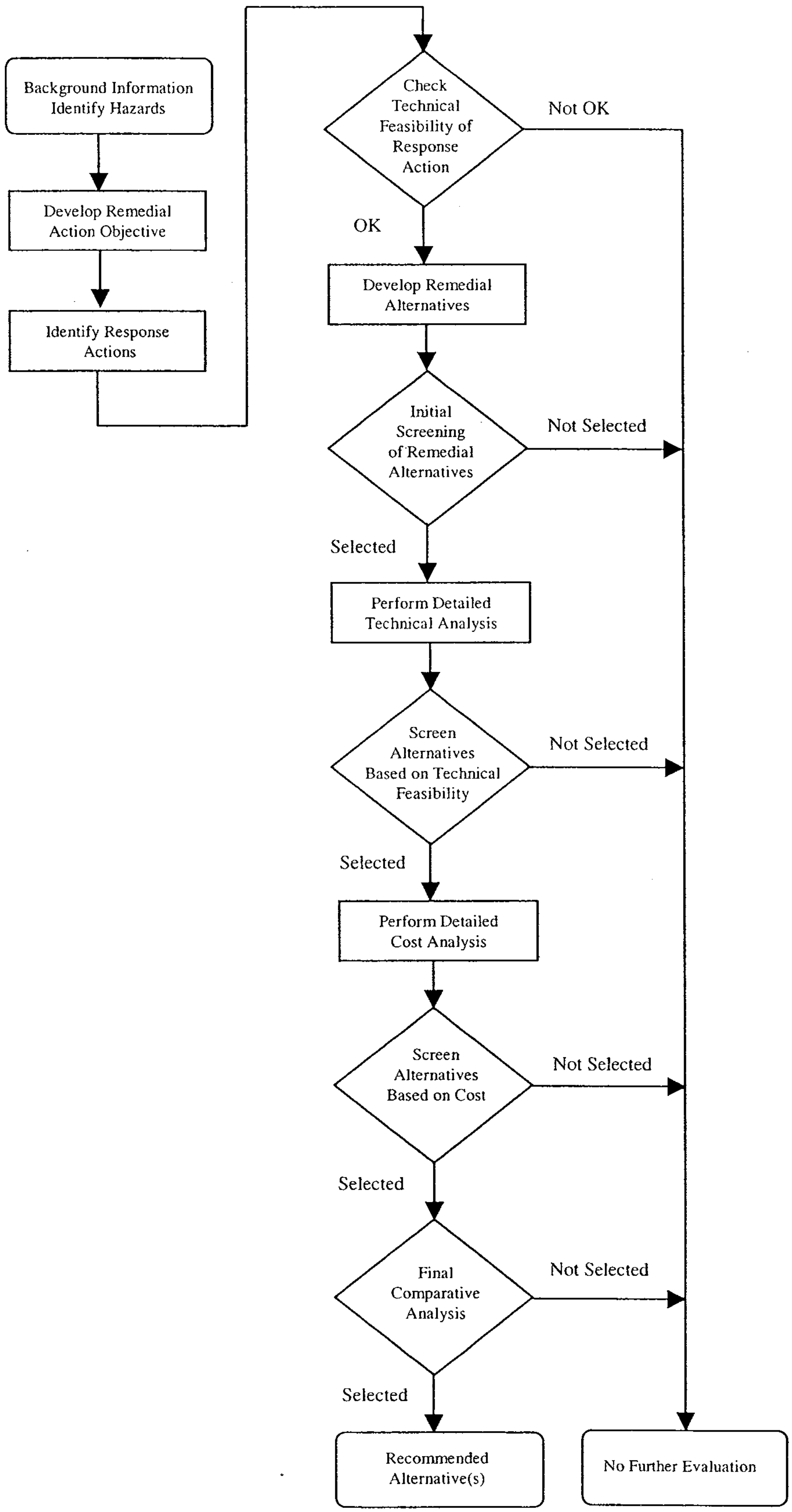


Figure 1-2

FEASIBILITY ANALYSIS FLOWCHART

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

westernmost end of Alameda Island, which lies on the eastern side of San Francisco Bay, adjacent to the city of Oakland. Alameda Point was occupied by the 1,734-acre former NAS Alameda until its closure in 1997. The Navy's intent is to turn over the site to the City of Alameda for possible conversion into a public golf course.

1.1.1 Site Description

IR Site 1 (and OU-3) encompasses approximately 78 acres. San Francisco Bay borders the site to the north and west.

IR Site 1 is relatively flat with slight depressions that sometimes flood during the winter rains. The site was previously used as a waste disposal site.

A portion of Runway 13 runs northwest-southeast through the site. There are a few uninhabited buildings and building foundations, a former picnic area, and a softball field located in the southern portion of the site. A former small arms range is located near the center of the western border (Figure 1-3). There are several paved roads that run through the site. Public access to IR Site 1 is currently restricted.

IR Site 1 was used for waste disposal at former NAS Alameda from 1943 to 1956. Prior to 1940, early maps show that the disposal area at IR Site 1 was under water (San Francisco Bay) at a depth of approximately 20 feet along the current western shoreline of the site. This area was reclaimed by dredging operations, which involved the placement of sunken barges and pontoons on the western edge of the disposal area, and clay and silt sediments in the disposal area. These operations are visible in aerial photographs taken in the 1940s. A jetty was later transformed into a seawall protecting the harbor entrance, which is now the northern edge of the disposal. New taxiways and runways were extended over the disposal area in the 1950s.

Information regarding the history of landfill contents is limited. The primary method used by NAS Public Works to dispose of wastes was to bulldoze trenches to the water table, fill with waste, and then compact the surface. In the early years of operation, the waste was simply pushed into the water. There are no records of placement of any liners in the landfill. Final cover material was applied to the landfill in later years.

Accurate estimates of the types and amounts of wastes deposited at IR Site 1 over the years are not available, but are believed to be approximately 15,000 to 200,000 tons of assorted refuse and debris, including scrap metal, waste oil, aircraft engines, low-level radiological wastes, solvents, paint wastes, cleaning compounds, creosote, waste medicines, reagents, asbestos, pesticides, mercury, and construction debris. Other naval installations, including Oak Knoll Naval Hospital, Naval Supply Center Oakland, and Treasure Island, also used the site for waste disposal (TtEMI, 1999; Ecology and Environment, 1983).

Geology

As described in the RI Report Addendum, Volume III (FWENC, 2002), subsurface soil conditions at the project site can be roughly characterized as Strata I through IV.

Stratum I

The fill comprising most of the site occurs between an elevation +6 and -10 mean sea level (msl) and is composed of mixtures of sand, silt, and clay dredged from the surrounding bay. Existing fill is mostly classified as SP (poorly graded sand), SP-SM (poorly graded sand with silt), with lean clay, gravel, and trash. The average moisture content and dry unit weight are 19 percent and 108 pounds per cubic feet (pcf), respectively. The average percent passing through a No. 200 sieve is 7 percent. The average standard penetration test (SPT) blow count ($N1_{(60)}$) is 13 blows per foot (bpf).

Stratum II

This unit consists generally of a very dark gray clay with varying amounts of sand and silt and marine shells and organic materials. This unit is commonly referred to as Young Bay Mud. Based on the available field and soil laboratory test data, this unit can further be divided into two subunits, namely the Offshore Bay Mud unit (Stratum IIA) and the Upland Bay Mud unit (Stratum IIB). The Stratum IIA predominantly consists of very soft fat clay (CH) and silt with high plasticity. The Stratum IIB in the site area predominantly consists of soft to medium stiff lean clay and silty clay/clayey silt. The thickness of Stratum IIA (offshore) is about 30 to 35 feet, and the thickness of Stratum IIB (upland) in the site area is about 15 to 25 feet. Stratum IIB is classified as sensitive fine-grained soils and is subject to strength degradation after cyclic loading (for example, earthquake loading). Stratum IIA (offshore) is expected to be as sensitive as Stratum IIB (upland).

Stratum III

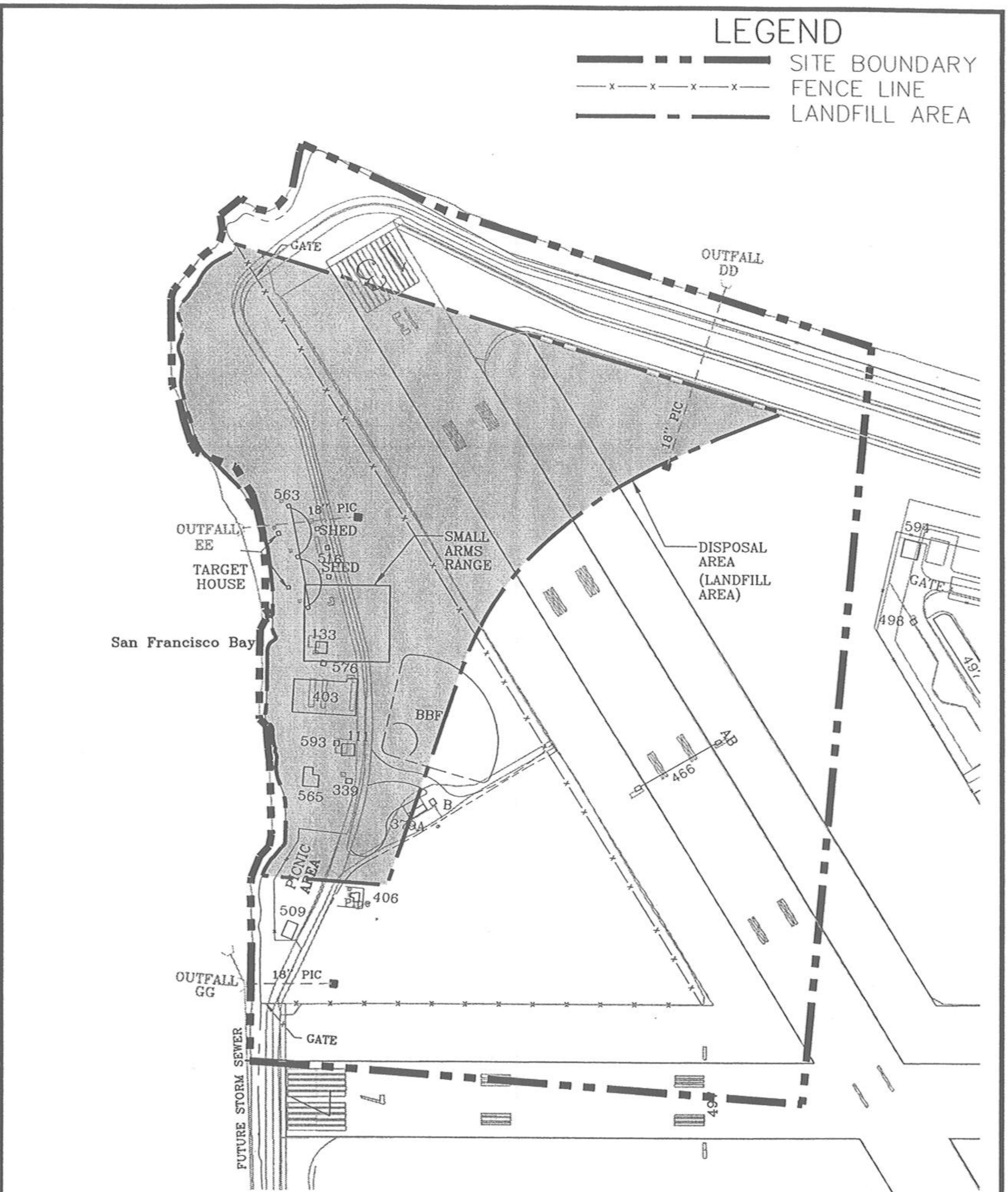
This unit comprises the Merritt Sand, mostly classified as dense fine-grained sand, SM and SP-SM, having an average moisture content and dry unit weight of 20 and 110 pcf, respectively. This layer occurs between elevations -22 to -76 msl. The average percentage passing through a No. 200 sieve is 16 percent.

Stratum IV

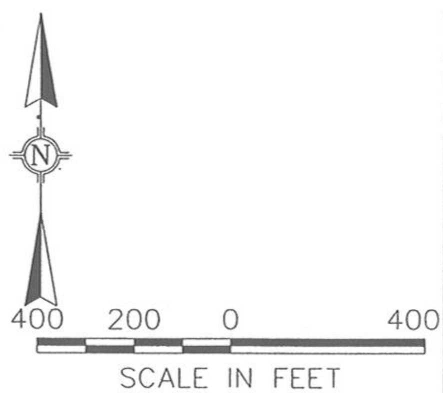
The Old Bay Mud in the vicinity of Alameda Point consists of stiff to hard, dark greenish-gray, very plastic silty clay. The clay occurs at a minimum elevation of -76 feet msl.

A summary of the geotechnical design parameters for each geologic unit is presented in Table 1-1. Information provided in the table includes: 1) available field data, 2) classification and index properties, and 3) engineering properties.

DRAWING NO: 03136813.DWG
DCN: FWSD-RAC-03-1368
CTO: # 0054
APPROVED BY: AL
CHECKED BY: TL
REV: REVISION 0
DRAWN BY: MD
DATE: 09/26/03



SOURCE:
OU-3 REMEDIAL INVESTIGATION REPORT, FINAL BY TETRA TECH EM INC., PUBLISHED IN RANCHO CORDOVA IN 1999.



I:\1990-RAC\CTO-0054\DWG\031368\03136813.DWG
PLOT/UPDATE: SEP 12 2003 15:34:24

Figure 1-3
IR SITE 1 SITE PLAN
ALAMEDA, CA

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

TABLE 1-1

SUMMARY OF GEOTECHNICAL DESIGN PARAMETERS

Generalized Stratum	**Units**	**I**	**IIA**	**IIB**	**III**	**IV**
Description		**Fill Materials**	**<u>Offshore</u> Soft Harbor Sediments, Young Bay Mud**	**<u>Upland</u> Soft Harbor Sediments, Young Bay Mud**	**Dense Sands**	**Stiff Clays**
Unified Soil Classification		SP, SP-SM, with CL, gravel and trash	Normally Consolidated (NC) to Slightly Underconsolidated (UC) fine-grained soils: ML, MH, CL, CH	Normally Consolidated (NC) fine-grained soils: ML, MH, CL, CH	SP, SP-SM, SM	CH
Borings providing data	No.	B1- B5, B10, B11	B6 through B10	B1 through B5, B11	B1 through B11	B2 and B4
Typical Elevation Range	feet msl	20 to + 10	40 to – 10	40 to – 10	35 to –75	Below – 75
Typical Thickness	feet	20 to 30	15 to 30	15 to 30	45 to 55	> 10
Raw SPT-N Values - Mean + Std. Deviation (No. data)	bpf	17 ± 12 (17)	2 ± 3 (29)	2 ± 3 (29)	> 50 ± 19 (60)	15 ± 4 (6)
Raw CPT Tip Resistance (Q_c) Values	tsf	18 ± 9 (38)		5.6 ± 2.1 (58)	200 ± 70	- - -
<u>Volumetric/Gravimetric Relationships</u>						
Total Unit Weight	pcf	128	100	115	131	125
Moisture Content	%	19	61	38	20	45
Dry Unit Weight	pcf	108	62	83	110	86
Void Ratio		0.58	>1.00	1.00	0.59	0.99
Specific Gravity		2.65	2.75	2.75	2.65	2.75
<u>Atterberg Limits</u>						
Liquid Limit, LL (AVG)	%	NP	55	43	NP	76
Plastic Limit, PL (AVG)	%	NP	26	21	NP	44
Plasticity Index, PI (AVG)	%	NP	29	22	NP	32
Liquidity Index, LI	%	NP	1.2	0.8	NP	.17
<u>Gradation Characteristics</u>						
Fines Content (< 74 microns), FC	%					
Clay Content (< 2 microns), CC	%					
Clay Activity Index, CAI						
<u>CD Shear Strength Parameters - Static Stability</u>						
Peak Internal Friction Angle (CD)	degrees	32		25	38	
Peak Cohesion Intercept (CD)	psf	0		0	0	
Residual Internal Friction Angle (CD)	degrees	30		25	38	0
<u>CU Shear Strength Parameters - Seismic Stability</u> (Pseudo-Static)						
SHANSEP's Normalized Static Pre-EQ Undrained Shear Strength $(Su/\Phi_v)'_{NC}$			0.2 (S_U = 300 psf)	0.2 (S_U = 500 psf)		0.3 (S_U = 1,300 psf)
SHANSEP's Normalized Post-EQ Undrained Shear Strength $(Su/\Phi_v)'_{NC}$			0.16	0.16		0.24
Post-Earthquake/Liquefaction Undrained Shear Strength $(S_U)_r$	psf	300	150	400		1,000 psf

TABLE 1-1 (Continued)

SUMMARY OF GEOTECHNICAL DESIGN PARAMETERS (SOURCE: FWENC, 2002)

Generalized Stratum	Units	I	IIA	IIB	III	IV
Description		Fill Materials	Offshore Soft Harbor Sediments, Young Bay Mud	Upland Soft Harbor Sediments, Young Bay Mud	Dense Sands	Stiff Clays
Compressibility Characteristics						
Over Consolidation Ratio, OCR						
Compression Index, Cc		0.08			0.025	- - -
Swelling Index, Ccs		0.020			0.006	
Recompression Index, Cr		0.016			0.005	
Coefficient of Consolidation, Cv	feet/year^2					18

Notes:

(Source: FWENC, 2002)

bpf – blows per foot
CPT – cone penetrometer test
FWENC – Foster Wheeler Environmental Corporation
msl – mean sea level
pcf – pounds per cubic foot
psf – pounds per square foot
SHANSEP – Stress History and Normalized Engineering Properties
SPT – standard penetration test
S_u – Undrained shear strength, used for end-of-construction stability evaluations
$(S_u)_r$ – Residual undrained shear strength, used for static post-earthquake stability evaluations
S_u/Φ_v – Undrained shear strength ratio, where σ'_{vo} is the initial effective overburden pressure
tsf – tons per square foot
Φ' – Effective internal friction angle

1.1.2 Remedial Investigation Report Addendum, Volume III, Findings

A geotechnical characterization (FWENC, 2002) of the site was performed in accordance with the requirements of the Final Focused RI Work Plan (FWENC, 2001). Field work began on December 5, 2001, and was completed on January 6, 2002. Field exploration consisted of performing 14 cone penetrometer tests (CPTs), excavating eight test pits, and drilling 11 soil borings. Results of field exploration were used to evaluate the existing condition of cover soils and to identify seismic hazards at the site.

Thickness of the cover soil varied from 6 inches to 2.5 feet. The existing soil cover was found to be inconsistent, poorly compacted, and very permeable. Because of these conditions, the material was determined to be unsuitable for use as part of the final cover design.

The seismic hazards identified at IR Site 1 included liquefaction potential and seismic slope instability. An integrated CPT-based method (Robertson and Wride, 1997) was used to quantify the potential for liquefaction and to identify areas susceptible to liquefaction. Based on the analyses, the upper fill material at the site exhibited a high potential for liquefaction and was designated as liquefiable. Liquefaction-induced settlements are estimated to be up to 12 inches. In addition to ground settlements, liquefaction-induced lateral spreading was estimated to be greater than 20 feet and much higher in some areas (up to 260 feet) using the empirical method proposed by Bartlett and Youd (1992) (revised by Youd, et al., 2002).

Different cross sections at the site were analyzed for stability. The program, PC-STABL-5M (Achilleos, 1988), based on limit equilibrium theory, was used to obtain factors of safety against slope failure. All cross sections analyzed had static factors of safety above 1. An extensive seismic hazard analysis was performed to obtain the peak horizontal ground acceleration (PHGA) and representative earthquake ground motion time histories at the site. The PHGA is the largest (absolute) value of horizontal acceleration recorded or expected at the site. Using Newmark-type procedures (Newmark, 1965), permanent lateral displacements at the site were obtained. Based on preliminary findings, predicted deformations are relatively high, ranging from 2 to 19 feet.

This FS was conducted to identify the most appropriate means to address the slope instability and liquefaction potential concerns and major hazards related to these concerns (for example, seismically induced large lateral displacements of the site perimeter slopes). The FS would evaluate various alternatives to mitigate the geotechnical and seismic hazards. These alternatives may include the following:

- Increasing seismic stability in the site area by stabilizing and increasing the shear strength of the Young Bay Mud (Stratum II) by in situ mixing with cement.
- Dredging and replacement of Young Bay Mud adjacent to the shoreline with stable quarry and rockfill materials.

- Providing stone columns to reinforce Young Bay Mud and accelerate its consolidation, and to minimize liquefaction potential of the upper fill layer by densifying granular soils and enhancing dissipation of excess pore pressures induced by earthquake.
- Minimizing lateral displacement and containing the potential contaminants from leaking into the bay by installing physical containment barriers along the shoreline (perimeter of the site).

1.1.3 Design Basis

The RI Report Addendum, Volume III (FWENC, 2002), reviewed the general constraints applicable to landfills and summarized the following geotechnical/seismic design basis applicable to future development and landfill closure activities at IR Site 1.

No formal classification has been established for landfills at either IR Site 1 or IR Site 2 as of this time. However, the Regional Water Quality Control Board (RWQCB) has indicated that IR Site 2, an area just south of IR Site 1, should be designated as a Class II waste management unit (landfills for designated waste). California Code of Regulations (CCR), Title 27, requires that Class II landfills be designed to the maximum credible earthquake (MCE). Additionally, the CCR requires that Class III landfills (landfills for non-hazardous solid waste) must be designed to the maximum probable earthquake (MPE). Title 22, which governs seismic and precipitation design standards for hazardous waste landfills (Class I), was not determined to be applicable for IR Site 1, and therefore, there was no reference to Title 22 in this report. However, the proposed seismic design of the IR Site 1 landfill closure satisfies Title 22 requirements specifically pertaining to MCE or seismic design (see Section 66264.25 of CCR Title 22). In general, the MCE results in a larger earthquake than the MPE. Therefore, as a conservative measure, it was decided to use the MCE as a basis for the seismic stability evaluations of IR Site 1.

For seismic stability, a pseudo-static factor of safety greater than 1.0 implies that slopes will not yield and remain stable. However, CCR Title 27, Section 21750(f)(5), requires that the pseudo-static factor of safety be equal to or greater than 1.5 when designing for the PHGA. CCR Title 27 adds that in lieu of achieving a factor of safety of 1.5 under dynamic conditions, a more rigorous analytical method that provides quantified estimate of the magnitude of movement (such as seismically induced slope deformation) may be used. When the pseudo-static factor of safety is less than 1.0, the slope yields and seismically induced permanent displacements will occur. Current engineering practice is to calculate the seismically induced displacements of the landfill slopes using a Newmark-equivalent method (Newmark, 1965; Seed and Bonaparte, 1992). For lined landfills, the allowable seismically induced slope displacements along liners are commonly set to a maximum of 6 inches to 1 foot. For unlined disposal facilities, there are no published standards or prescribed maximum values for allowable seismically induced slope displacements.

For cover soil systems, there is no maximum deformation specified. Regulations simply indicate that the cover system must "withstand earthquake loading." However, because cover repairs can be made more easily than liner repairs, current practice is to allow a greater level of deformation and that is to be evaluated on a case-by-case basis.

For IR Site 1, since it is an unlined disposal facility and is planned to be converted into a golf course, larger permanent seismically induced slope displacements on the order of several feet may be allowed. Selection of a more precise value for the allowable seismic design displacement depends on the following factors:

1. Width of the buffer zone between the waste limit and the shoreline along San Francisco Bay on the west or the Oakland Inner Harbor channel on the north side of the site
2. The nature of the remediation measure(s) that may be used to limit the seismic displacements of the landfill perimeter slopes.

2.0 DEVELOPMENT OF REMEDIAL ACTION OBJECTIVE, RESPONSE ACTIONS, AND PERFORMANCE CRITERIA

In this section, a remedial action objective is developed to address the geotechnical and seismic hazards identified in the Remedial Investigation (RI) Report Addendum, Volume III [Foster Wheeler Environmental Corporation (FWENC), 2002]. General approaches or response actions are then identified, which will achieve the remedial action objective. Two main categories of response actions are discussed: soil improvement and installation of physical buttresses. Various remedial methods associated with each response action are identified, and performance criteria are developed to evaluate implementability or technical feasibility of the selected remedial methods. Because of the technical limitations associated with each method, satisfaction of the remedial action objective will require development of remedial alternatives based on a combination of these methods. A preliminary technical evaluation was performed to evaluate feasibility of the general approaches or response actions.

2.1 REMEDIAL ACTION OBJECTIVE

The remedial action objective was developed based on the following considerations: 1) future use of the site; 2) existing geotechnical and seismic hazards; and 3) other concerns such as low bearing capacity and poor hydraulic performance of the existing soil cover, differential settlements caused by the future landfill cap, and the impact of sunken barges and pontoons identified in the RI Report Addendum, Volume III (FWENC, 2002).

The City of Alameda has proposed that Installation Restoration (IR) Site 1 be used as a golf course after transfer from the Navy. Because golf course construction involves light structures and there are no other permanent installations or structures planned for the site, the risks associated with the effects of potential deformations of the disposal area are considered to be very low over most of the site. According to Resource Conservation and Recovery Act (RCRA) Subtitle D (258), seismic design guidance for municipal solid waste landfill facilities: "For cover systems, where permanent seismic deformations may be observed in post-earthquake inspections and damage to components can be repaired, larger permanent deformations may be considered acceptable. In fact, some regulatory agencies consider seismic deformations of the landfill cover system primarily a maintenance problem" [U.S. Environmental Protection Agency (EPA), 1996].

Geotechnical and seismic hazards identified in the RI Report Addendum, Volume III (FWENC, 2002), included static and seismic slope instability and liquefaction potential. These hazards can lead to relatively large seismically induced slope displacement and liquefaction-induced settlements and permanent lateral deformations. The magnitude of permanent lateral deformations due to slope instability was estimated to be up to 19 feet (RI Report Addendum, Volume III; FWENC, 2002). Because the site was formerly used as a landfill, a major concern is release of waste into San Francisco Bay.

Except for a 50- to 100-foot-wide zone parallel and adjacent to the shoreline, lateral deformations on the order of several feet may be considered acceptable because these localized lateral deformations can be addressed as a maintenance requirement (EPA, 1996).

Other concerns identified in the RI Report Addendum, Volume III (FWENC, 2002), included low bearing capacity and poor hydraulic performance of the existing soil cover, potential for future foundation settlements, and the impact of sunken barges and pontoons. It is anticipated that at least 4 feet of fill material will be placed as soil cap at the site. Bearing capacity failures are related to general rotation and heaving of soil mass. Since a relatively uniform fill will be applied throughout the site, bearing capacity failure potential is considered negligible. Localized bearing capacity failure and foundation settlements potential can be addressed in the final design of the cap. The hydraulic performance of the existing soil cover (permeability of the cover soils and its function as a liquid barrier) was found to be inconsistent and generally poor. However, this is not considered a major concern because an engineered soil cap is planned that will meet the applicable regulatory requirements, including hydraulic performance. Concerns regarding differential settlements will be addressed in the design of the landfill cap, which will provide for a positive drainage over the cap and minimize potential ponding due to these settlements. The impact of sunken barges and pontoons was found to be negligible since they were placed away from the predicted failure surfaces (RI Report Addendum, Volume III; FWENC, 2002).

For the purposes of this Geotechnical Feasibility Study (FS) Report, the remedial action objective is to prevent release of waste into San Francisco Bay by increasing slope stability and reducing potential lateral deformations. The slope deformations and settlement values are not restricted within the site and may extend beyond the site boundary. However, as indicated, the remedial action objective is to prevent release of waste from the site into San Francisco Bay during seismic activities. The objective of remedial measures is not necessarily to preserve the golf course or adjacent areas from seismic effects. Therefore, the focus of the proposed remedial measures will be to control release of waste into San Francisco Bay and to address the geotechnical and seismic hazards identified within the boundaries of IR Site 1.

2.2 GENERAL RESPONSE ACTIONS

Possible remedial methods available to mitigate the geotechnical and seismic hazards can be classified under two types of general approaches or response actions. These response actions include implementation of soil improvement and installation of physical buttresses along the shoreline perimeter of the site. Both response actions address the general intent of the remedial action objective and are discussed in more detail in the following sections.

2.2.1 Soil Improvement

The soil conditions summarized in Section 1.2 indicate that a weak Young Bay Mud layer exists at IR Site 1. Results of geotechnical laboratory testing show this layer to be slightly underconsolidated. Soils that are underconsolidated are considered highly compressible since they are still undergoing settlements under their own weight and the existing overburden load (soil cover) and also tend to have low shear strength properties, which affect slope stability. Since the predicted failure surfaces are expected to develop within this soil layer, the properties of the Young Bay Mud have a significant effect on overall slope stability as well as the magnitude of seismically induced lateral deformations. The engineering properties of the Young Bay Mud soil layer can be improved by implementing soil improvement methods that can either accelerate consolidation of this layer or by in situ mixing with cement. Both methods are expected to increase the shear strength of the soil, resulting in increased slope stability and reduced lateral deformations.

Excavation of weak soil and refuse and replacement (backfilling) with clean and well-compacted material over the entire site can also improve soil conditions. This method involves removal of potentially impacted soil from the liquefiable fill layer and/or weak Young Bay Mud layer and backfill with clean imported fill material. However, this method is not considered practical and is not included in this Geotechnical FS Report due to various disadvantages. The key concerns are related to potential exposure to the environment and human health associated with handling of a very large quantity of waste.

Soil improvement methods considered for this Geotechnical FS Report included: 1) installation of wick drains, 2) application of surcharge (additional fill placement for consolidation), 3) installation of stone columns, 4) installation of a soil cement gravity wall, 5) excavation along shoreline and soil backfill, and 6) partial solidification. Descriptions of each of the soil improvement methods are presented in Section 3.1.

2.2.2 Physical Buttresses

Physical buttresses are commonly installed at sites to address slope stability problems. This response action would involve placement of retaining structures along the edge of the bay interface at the site and could extend outward into the bay or inward toward the site. Buttresses are designed to increase the slope stability factor of safety and decrease lateral movement by providing an additional resisting force to counter driving forces.

The types of physical buttresses considered for this Geotechnical FS Report included: 1) drilled concrete piers, 2) sheet piles, 3) soil bentonite cutoff wall, 4) riprap embankment with soil backfill, 5) inclined timber piles, 6) vibrated beam cement bentonite cutoff wall, 7) vibrated beam "Impermix" cutoff wall, 8) concrete wall, 9) pre-cast concrete piles, and 10) excavation with riprap. Descriptions of each type of physical buttress are presented in Section 3.1.

2.3 PERFORMANCE CRITERIA

The remedial action objective is to prevent release of waste into San Francisco Bay. It can be accomplished by increasing slope stability and/or reducing potential lateral deformations. Remedial alternatives developed to address the remedial action objective will be composed of one or more of the remedial methods listed in Sections 2.2.1 and 2.2.2, above. In this section, performance criteria are established to evaluate each remedial alternative for technical feasibility. The performance criteria are developed for both static and seismic loading conditions.

Static (long-term) slope stability is normally quantified by a factor of safety defined as the ratio of resisting (stabilizing) forces to the driving forces, the forces trying to horizontally move, or overturn the slope. A value of 1.0 or greater indicates that the slope is statically stable. However, because of the uncertainties associated with variability of soil conditions, measured soil shear strength, and limitations of analysis methods, the current state of practice in geotechnical engineering for static design is to require a minimum factor of safety of 1.5.

The effects of seismic loading can be quantified by calculating either a pseudo-static factor of safety when subject to a pseudo-static acceleration equal to the site design peak horizontal ground elevation (PHGA) or estimating the amount of permanent lateral displacements. A pseudo-static factor of safety greater than 1.0 is considered acceptable. Current engineering practice is to calculate the seismically induced displacements of the landfill slopes using a Newmark-equivalent method (Newmark, 1965; Seed and Bonaparte, 1992). Since there are no specific regulatory guidelines for unlined landfills that specify maximum allowable deformations, the allowable deformation criteria were based on site usage, remedial action objective, and performance criteria for similar types of applications.

A review of design/performance criteria for other types of earth structures, such as earth dams, indicated that a similar approach is used to arrive at the maximum allowable deformation performance criteria for earth dams (Beikae, 2002; Makdisi and Seed, 1978). Earth dams and landfills both act as containment systems, holding water and waste, respectively. Performance criteria for both structures limit the amount of deformation it can sustain to maintain functionality. The factors that define deformation performance criteria for earth dams include:

- The purpose and use of the dam
- Hazards/risks associated with failure of the dam
- Available freeboard distance for the dam
- Width of the dam drain and filter zones

These factors are similar to those discussed for establishing the performance criteria for the seismic stability evaluation of the IR Site 1 perimeter berm. Among the above factors, the available freeboard of a dam (a variable similar to the width of the buffer zone between the waste limit and shoreline at IR Site 1) is the most critical variable in establishing the seismic deformation performance criteria to avoid a catastrophic failure and ensure life safety. Freeboard for a dam represents a vertical safety distance that should be maintained to prevent overtopping. Similarly, a buffer zone between the waste limit and the shoreline should be maintained to prevent migration of contaminants toward San Francisco Bay. Based on the available freeboard, maximum seismically induced displacements of up to 15 feet and 3 feet representing horizontal components of movement along the failure plane were considered acceptable in the design of two large earth dams recently built in Southern California (Makdisi and Seed, 1978; Beikae et al., 1996).

For this Geotechnical FS Report, the amount of allowable lateral deformations will depend on the following factors:

- Width of the buffer zone between the waste limit and the shoreline along San Francisco Bay on the west, and the Oakland Inner Harbor Channel on the north side of the site
- The type of remedial alternative proposed to limit seismic deformations of the landfill perimeter slopes (This factor is a specific design requirement discussed in more detail in Section 4.2.2)

No waste delineation was performed as part of the work detailed in the RI Report Addendum, Volume III (FWENC, 2002). Therefore, the width of the buffer zone is unknown. However, based on the construction history and aerial photographs showing locations of disposal areas, it is estimated that the buffer zone is approximately 8 to 15 feet wide (Pacific Aerial Surveys 1949, 1957). In order to satisfy the remedial action objective of preventing waste release into San Francisco Bay, the allowable lateral displacement should be less than the minimum width of the buffer zone (8 feet). Since direct measurements of the width of the buffer zone are not available, an allowable lateral displacement of 4 feet was selected to provide an adequate safety margin (safety factor of 2). This allowable displacement selected is at the lower end of the range of allowable displacement criteria used for other types of earth structures such as earth dams (Makdisi and Seed, 1978; Beikae et al., 1996).

In addition to evaluating seismic stability during earthquake shaking, post-earthquake stability of slopes should also be evaluated using soil post-earthquake residual strength parameters (Ramanujam et al., 1978; FWENC, 2002). A post-earthquake static factor of safety greater than 1.0 is considered acceptable.

Based on the above discussion, the following performance criteria are used to establish technical feasibility for each remedial alternative proposed:

- Static factor of safety for site slopes should be at least 1.5.

- Pseudo-static factor of safety for site slopes when subjected to a pseudo-static acceleration equal to the site design PHGA should be greater than 1.0 or allowable seismic displacements should be limited to less than 4 feet.
- Post-earthquake static factor of safety should be greater than 1.0. The factor of safety decreases after an earthquake due to the residual shear strength values of liquefied materials resulting in a minimum factor of safety under static conditions. Subsequently, the static factor of safety increases since the liquefied materials become denser over time as a result of the consolidation process initiated by the weight of the materials.

2.4 PRELIMINARY TECHNICAL EVALUATION OF RESPONSE ACTIONS

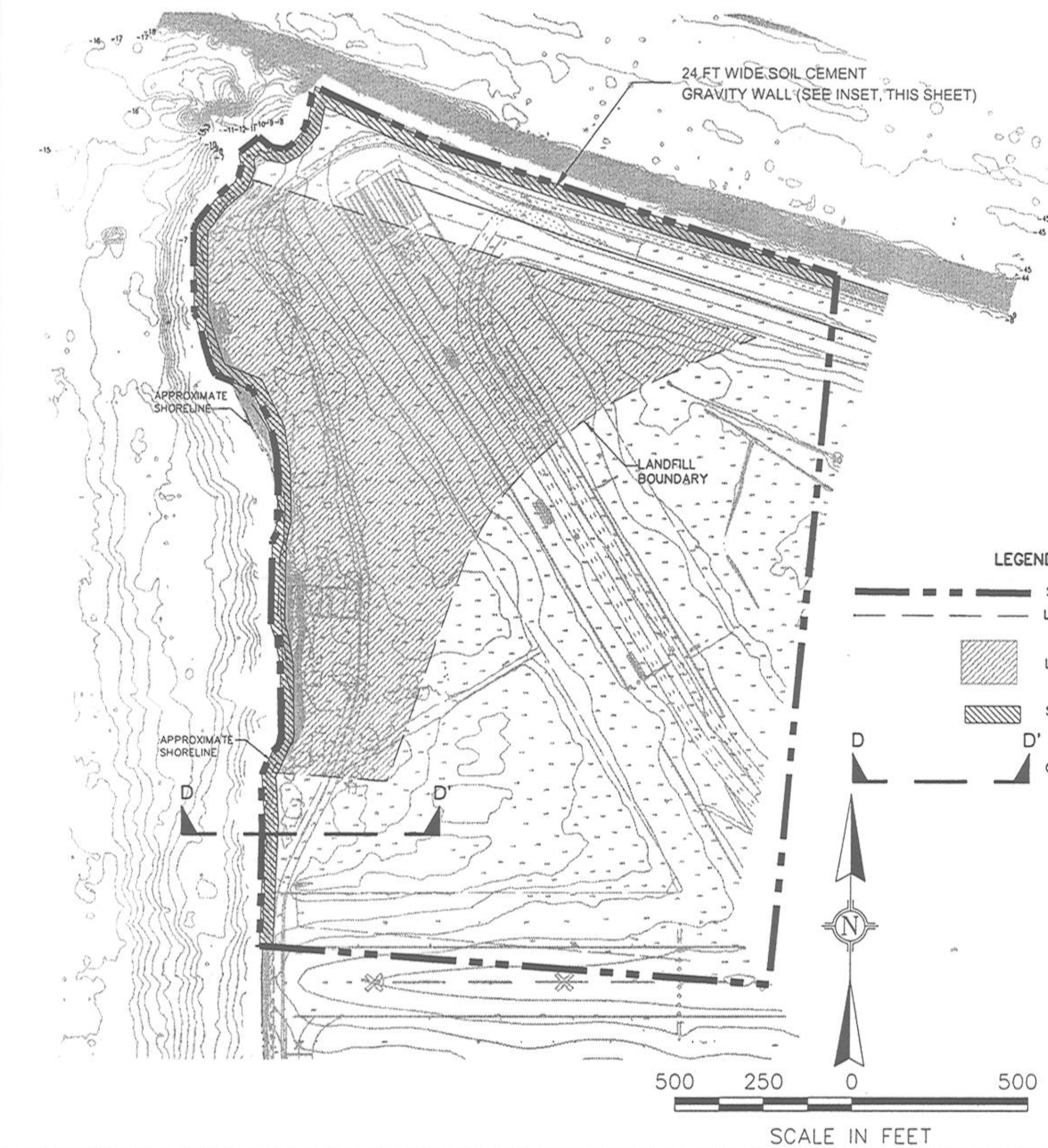
This section presents a summary of the preliminary technical evaluation of the two response actions: soil improvements and physical buttresses. This evaluation was performed to determine if the general response actions proposed to achieve the remedial objective are technically feasible. More detailed analyses performed are discussed in Section 4.0. The preliminary technical evaluation process consisted of building a two-dimensional slope cross section model for each type of the two response actions, calculating pre- and post-earthquake slope stability factors of safety, and estimating permanent lateral deformations. Stability analyses were performed using conventional two-dimensional limit-equilibrium methods. The computer program PC-STABL-5M (Achilleos, 1988) was used to calculate long-term static and post-earthquake static factors of safety. Lateral deformations were estimated using a Newmark-type double-integration method (Newmark, 1965). The results of these two models are discussed below.

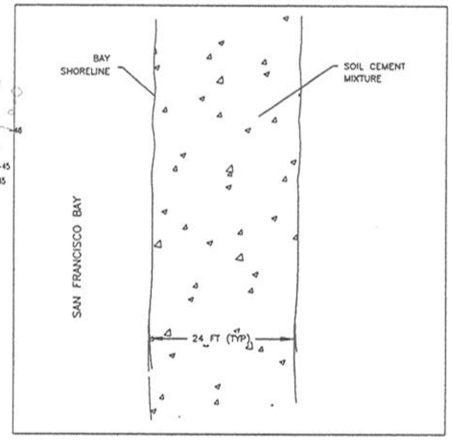
Soil Improvement Model

Figures 2-1 and 2-2 show a schematic plan view of the shoreline and a typical cross section at IR Site 1, respectively. Areas with proposed soil improvements are highlighted in both figures. Because the site slopes along its western and northern perimeters to San Francisco Bay and Oakland Inner Harbor, respectively, slope stability depends mainly on soils near the shoreline perimeter of the site. Therefore, soil improvements will generally be implemented across a narrow zone along the shoreline of the site and not over the entire area. The effectiveness of any soil improvements depends on the extent (width/depth) and type of soil improvement methods used. To evaluate the effectiveness of the soil improvement response action in general, preliminary technical evaluation involved analysis of typical soil improvement methods. In this case, a soil cement gravity wall was randomly selected and modeled. Parameters used in the model included a 24-foot-wide wall, which extended down to the bottom of the Young Bay Mud layer. The model also used higher shear strength soil parameters for areas where the soil cement wall was placed. Existing and proposed shear strength soil parameters modeled for each soil layer are shown in Figure 2-2. The amount of increase in soil shear strength depends on the type of soil improvement method selected.

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	03136821.DWG

I:\1990-RAC\CTO-0054\DWG\031368\03136821.DWG
PLOT/UPDATE: SEP 12 2003 15:35:01





INSET
(NOT TO SCALE)

LEGEND

- SITE BOUNDARY
- LANDFILL BOUNDARY
- LANDFILL AREA
- SOIL IMPROVEMENT
- D D' CROSS SECTION

REFERENCE:
HJW-GeoSpatial, Inc., Upland Topography
NAD27, NGVD29 - CCS Zone III

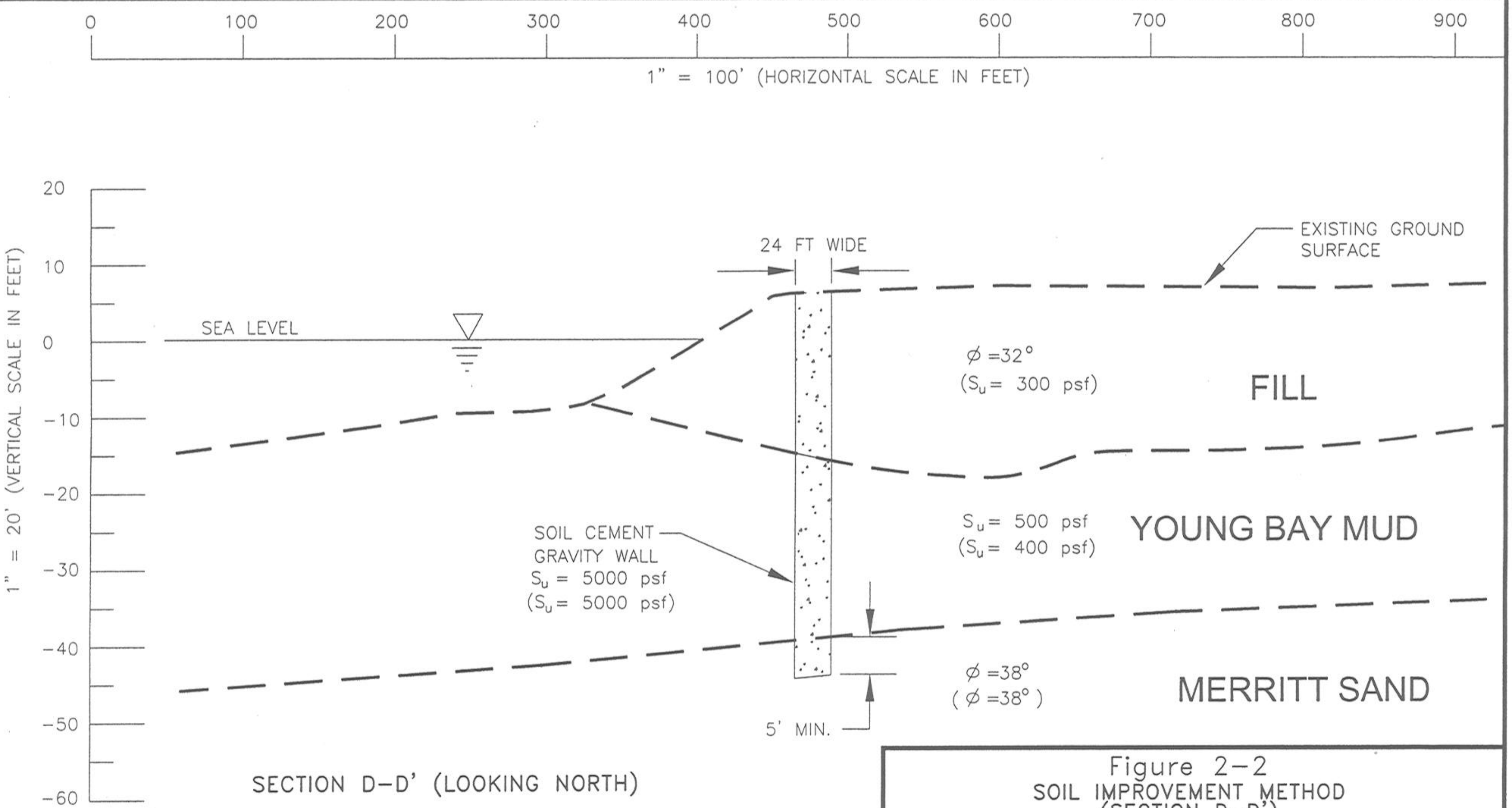
Figure 2-1
SOIL IMPROVEMENT METHOD
(PLAN VIEW)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWN BY: M.D.	CHECKED BY: TL	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 03136822.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	

I:\1990-RAC\CTO-0054\DWG\031368\03136822.DWG
PLOT/UPDATE: SEP 12 2003 15:42:33



SECTION D–D' (LOOKING NORTH)

Notes:
Shear Strength Parameters:
ϕ = Friction Angle
S_u = Undrained Shear Strength
Parenthesis ( ) indicate post-earthquake values
psf = Pounds per square foot

Figure 2–2
SOIL IMPROVEMENT METHOD
(SECTION D–D')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

Results of the stability analyses are summarized in Table 2-1. The minimum slope stability factors of safety and estimated lateral slope movements before and after soil improvements are provided in the table. Based on the analysis results, the estimated lateral slope movement before implementation of the response action is approximately 16 feet, well above the 4-foot limit established in the performance criteria. After soil improvements, the estimated lateral slope movement is reduced to 3 feet. The long-term static and post-earthquake static slope stability factors of safety were estimated to be 3.03 and 2.13, respectively, compared to 1.66 and 1.38 for the original slope conditions. These results demonstrate that the soil improvement response action is technically feasible and that other improvement methods associated with this action should be considered for further evaluation and screening.

TABLE 2-1

RESULTS OF STABILITY ANALYSIS

Case Analyzed	Long-Term Static Slope Stability Factor of Safety	Post-Earthquake Static Slope Stability Factor of Safety	Estimated Lateral Slope Movement (feet)
Original Slope Conditions	1.66	1.38	16
Slope with Soil Improvement	3.03	2.13	3
Slope with Physical Buttress	4.39	4.13	2.6

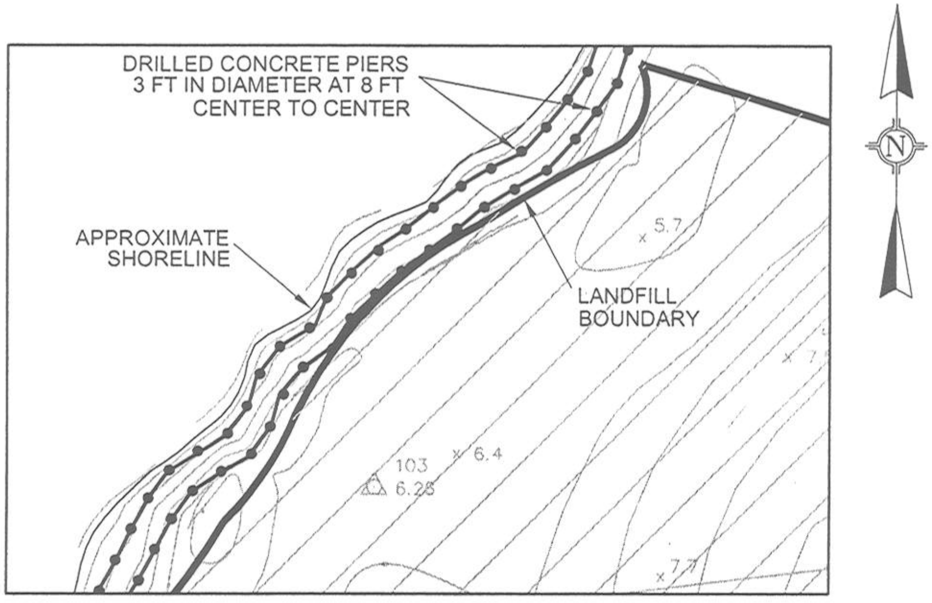
Physical Buttress Model

A similar approach, as discussed in the Soil Improvement Model section, was followed to perform a preliminary technical evaluation of the physical buttress response action. Figures 2-3 and 2-4 show a schematic plan view of the shoreline and a typical cross section at IR Site 1, respectively. This is the same area and cross section analyzed in the previous Soil Improvement Model. In this example analysis, a system of two rows of drilled concrete piers spaced 8 feet center to center along the shoreline and extending 20 feet into the Merritt Sand layer was randomly selected for analysis and was modeled. Shear strength parameters for each soil layer were assumed to be the same, before and after the physical buttress was installed.

Results of the stability analyses are summarized in Table 2-1. The minimum slope stability factors of safety and estimated lateral slope movements are provided for the original slope condition case and slope with the physical buttress. As in the previous evaluation, the results demonstrate that this (physical buttress) response action is technically feasible and that other types of physical buttresses should be considered for further evaluation and screening.

I:\1990-RAC\CTO-0054\DWG\031368\03136823.DWG
PLOT/UPDATE: SEP 12 2003 15:44:18

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	03136823.DWG



INSET - A

50 25 0 50
SCALE IN FEET

LEGEND

- SITE BOUNDARY
- LANDFILL BOUNDARY
- LANDFILL AREA
- DRILLED CONCRETE PIER
- D D' CROSS SECTION

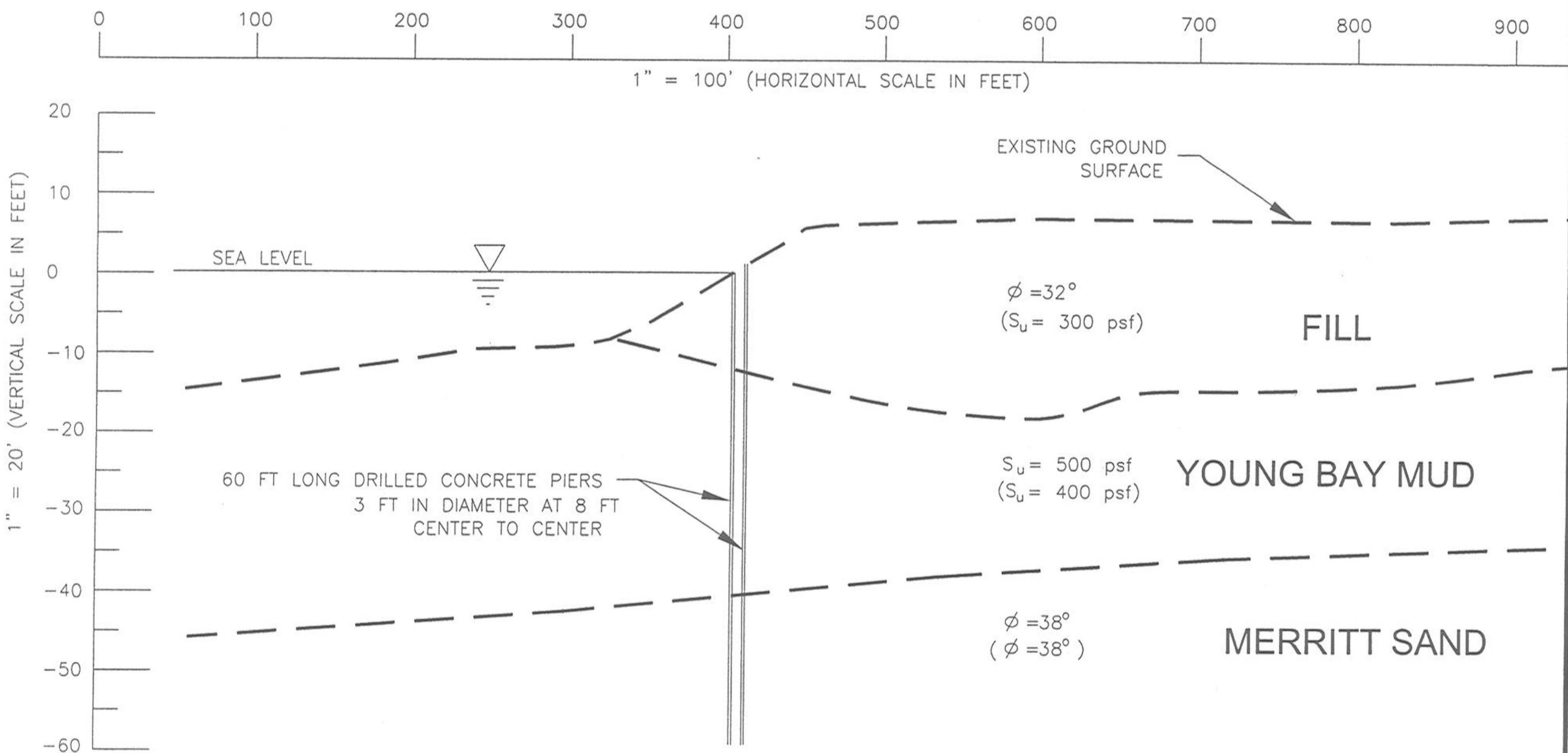
REFERENCE:
HJW-GeoSpatial, Inc., Upland Topography
NAD27, NGVD29 - CCS Zone III

Figure 2-3
PHYSICAL BUTTRESS METHOD
(PLAN VIEW)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWN BY: M.D.	CHECKED BY: TL	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1348	DRAWING NO:
DATE: 09/26/03	REV.: REVISION 0		CTO: # 0054	03134824.DWG



SECTION D-D' (LOOKING NORTH)

Notes:
Shear Strength Parameters:
- ϕ = Friction Angle
- S_u = Undrained Shear Strength
- Parenthesis ( ) indicate post-earthquake values
- psf = Pounds per square foot

Figure 2-4
PHYSICAL BUTTRESS METHOD
(SECTION D-D')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\03136824.dwg 09/12/2003 03:45:10 PM PDT

3.0 DEVELOPMENT AND SCREENING OF ALTERNATIVES

The response actions identified in the previous section included soil improvement and installation of physical buttresses around the perimeter of the site. Preliminary evaluation of the response actions indicated that the remedial action objective can be satisfied by implementation of either of these actions. Specific types of soil improvement methods and physical buttresses were also identified in the previous section. In this section, remedial alternatives are developed through the soil improvement method, the physical buttress method, and by combining the two. The individual and combined remedial alternatives will meet the performance criteria. A brief description of each remedial alternative considered is provided, followed by an outline of the evaluation criteria used for screening of alternatives. Results of initial screening performed are summarized, and selected remedial alternatives that will undergo detailed analysis in Section 4.0 are identified.

3.1 DEVELOPMENT OF ALTERNATIVES

Soil improvement methods and types of physical buttresses were proposed in Sections 2.2.1 and 2.2.2 in order to increase slope stability and decrease lateral displacements under static and seismic loading. Some of these methods considered individually may not satisfy the established performance criteria. In order to satisfy the remedial action objective and to meet the established performance criteria, remedial alternatives were developed by combining individual soil improvement and physical buttress methods.

Table 3-1 lists 20 remedial alternatives developed from specific response action methodologies. Each alternative can be classified as a soil improvement, a physical buttress, or a combination of both methods. In general, soil improvements would be made only in relatively narrow areas along the site shoreline perimeter affecting the potential slope failure surface, as shown in Figure 3-1. The physical buttresses would be installed along the western and northern perimeters of the site bordering San Francisco Bay and the Oakland Inner Harbor Channel, respectively. Each alternative addresses the established performance criteria of preventing waste release into the San Francisco Bay/Oakland Inner Harbor Channel by reducing lateral spreading due to liquefaction, minimizing lateral slope movements due to slope instability, or by a combination of both. Table 3-1 shows the primary hazard that is mitigated by each alternative. Brief descriptions of each remedial alternative are provided in this section. A more detailed description of the selected alternatives is presented in Section 4.0.

TABLE 3-1

PROPOSED ALTERNATIVES

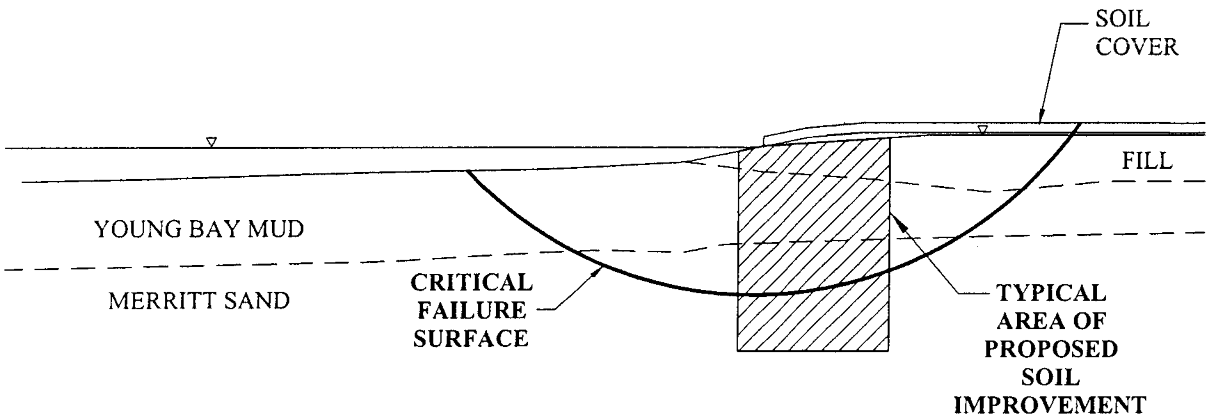
Alternative No.	Description	Type of Response Action			Primary Hazard Addressed*
		Soil Improvement	Physical Buttress	Combined Method	
1	Wick Drains with Surcharge	X			S
2	Stone Columns with Surcharge	X			L
3	Sheet Piles with Anchors		X		S
4	Stone Columns with Surcharge and Sheet Piles			X	B
5	Soil Cement Gravity Wall and Stone Columns			X	B
6	Concrete Wall		X		B
7	Excavation with Riprap		X		B
8	Drilled Concrete Piers with Stone Columns			X	B
9	Pre-cast Concrete Piles		X		S
10	Wick Drains with Surcharge and Sheet Piles			X	B
11	Excavation along Shoreline and Soil Backfill	X			B
12	Partial In Situ Solidification	X			B
13	Soil Bentonite Cutoff Wall		X		B
14	Riprap Embankment in the Bay and Soil Backfill	X			S
15	Inclined Timber Piles		X		S
16	Consolidation with Surcharge	X			S
17	Wick Drains with Vacuum	X			S
18	Vibrated Beam Cement Bentonite Cutoff Wall		X		B
19	Vibrated Beam Impermix Cutoff Wall		X		B
20	Soil Cement Gravity Wall			X	B

Notes:

* B – both liquefaction and slope instability
L – liquefaction/lateral spreading
S – slope instability

I:\1990-RAC\CTO-0054\DWG\031368\03136831.DWG
PLOT/UPDATE: SEP 12 2003 15:46:44

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1348	DRAWING NO: 03134831.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 3-1
TYPICAL SOIL IMPROVEMENT AREA RELATIVE TO POTENTIAL FAILURE PLANE

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

Alternative 1: Wick Drains with Surcharge

This first remedial alternative is a soil improvement method that would include installation of wick drains with surcharge application. Wick drains, which consist of a vertical polypropylene core wrapped in a jacket, would be hydraulically pushed or vibrated into the ground. The high horizontal permeability of the jacket allows water to enter the wick drain while simultaneously filtering out soil particles. Wick drains would be installed along narrow areas along the shoreline perimeter of the site extending down to the Young Bay Mud layer. Short drainage paths would be provided by installing wick drains, which would accelerate the process of consolidation of the Young Bay Mud layer. This condition would lead to an increase in the shear strength of the soil and provide increased slope stability while reducing lateral deformations during seismic events.

Clean fill material would be applied as a surcharge and to provide additional overburden pressure to further accelerate consolidation.

Alternative 2: Stone Columns with Surcharge

This soil improvement method would also involve accelerating the process of consolidation in the Young Bay Mud layer. Soil borings are performed along the shoreline perimeter of the site extending down to the Young Bay Mud layer. The boreholes are then filled with stones to act as a filter and provide a vertical drainage path. Surcharge loading is applied by placement of clean fill material over the site to further accelerate the consolidation process.

In addition to increasing the shear strength of the Young Bay Mud layer through consolidation, the installation of stone columns would replace existing weak soils (fill material and Young Bay Mud) with higher strength stones (increased shear strength) and densify the surrounding soils. Commonly used methods to install stone columns include the following:

- **Wet top feed stone column method:** This method uses a vibratory probe inserted into the ground. It is a wet method because pressurized water is used to penetrate to the required depth. Once the vibratory probe reaches the desired depth, gravel is added from the ground surface and compacted as the vibratory probe is pulled up.
- **Dry bottom feed stone column method:** This method is similar to the wet top feed stone method, except that vibration with compressed air is used to penetrate the ground and reach the required depth. Also, the gravel (stone) is inserted through a separate tube alongside the vibratory probe (hence, bottom feed). As with the wet top feed stone column method, the stones are compacted in several lifts as the vibratory probe/bottom feeder is pulled up.
- **"Franki" stone column method:** This method developed by Frankipile Australia (part of the Keller Group, a leader in ground improvement engineering based in the UK) uses steel tubes driven into the ground. A temporary steel liner tube is driven to the required depth. The stones are then added from the top of the tube and driven out using a drop hammer as the tube is raised.

Appropriate methods for installing stone columns will be further evaluated during the detailed design phase.

Alternative 3: Sheet Piles with Anchors

In this remedial alternative, steel sheet piles would be installed along the shoreline perimeter as a physical buttress. Sheet piles consist of interlocking steel members driven deep into the soil to form a containment system. The function of the sheet piles is to contain any waste material that could be released during a seismic event. Sheet piles are generally weak in bending (flexible) because they are slender. Therefore, the sheet piles would have to be driven deep into the Merritt Sand layer for stability. In addition, anchors (tension structural members) would be installed with the sheet piles to minimize lateral deflections. Anchors, which are normally made of steel, would be driven into the soil to provide additional support for the sheet piles.

Alternative 4: Stone Columns with Surcharge and Sheet Piles

This alternative is a combination of Alternatives 2 and 3. Stone columns with fill surcharge would be installed adjacent to the sheet pile to accelerate the consolidation of the Young Bay Mud layer, while the steel sheet piles would provide a containment system around the shoreline perimeter. The surcharge loading is applied by placement of clean fill material. Additionally, stone columns would reduce the liquefaction potential in the granular soils of the upper fill layer (Stratum I) by densifying the soil and reducing excess pore water pressure. The fill surcharge would consist of the same material as required for the landfill cap. After full consolidation of the Young Bay Mud layer, the fill surcharge would then be used as landfill cap material.

Alternative 5: Soil Cement Gravity Wall and Stone Columns

This alternative is partially a soil improvement method, which results in the creation of a physical buttress. Large-diameter augers are used to inject cement slurry within a narrow zone along the shoreline perimeter of the site extending down to the bottom of the Young Bay Mud layer. Cement slurry mixes with existing soil material, which would form stabilized blocks or columns of soil. The resulting interlocking barrier is commonly described as a soil cement gravity wall. This gravity wall is supplemented by installing stone columns within the overlying fill material.

Alternative 6: Concrete Wall

This alternative is partially a soil improvement method, which results in the creation of a physical buttress. The concrete would be installed within a narrow zone along the shoreline from the ground surface to the bottom of the Young Bay Mud layer. A trench would be excavated using a slide rail system to hold up the sides of the excavation. The excavation would be constructed in sections and backfilled with ready mix concrete. In order to eliminate off-site disposal costs, the excavated material would be placed in the existing landfill area and temporarily capped with 2 feet of fill material.

Alternative 7: Excavation with Riprap

This alternative consists of excavating the top fill material and the Young Bay Mud within a narrow zone along the shoreline perimeter extending offshore. The width of the excavation would depend on previously predicted failure surfaces [Foster Wheeler Environmental Corporation (FWENC, 2002)]. Excavated material would be replaced with riprap, which would act as a physical buttress in stabilizing the slopes along the shoreline.

Alternative 8: Drilled Concrete Piers with Stone Columns

This remedial alternative consists of installing two rows of concrete piers along the shoreline perimeter of the site extending below the Young Bay Mud layer into the Merritt Sand layer. Drilled concrete piers would be installed from the existing ground surface to 60 feet deep. Two rows of evenly spaced concrete piers would be drilled at the shoreline and backfilled with concrete. The arrangement forms a physical buttress. In addition, stone columns would be installed in the fill layer between the two rows of drilled concrete piers to minimize the effect of liquefaction.

Alternative 9: Pre-cast Concrete Piles

This remedial alternative consists of installing four rows of pre-cast concrete piles along the shoreline perimeter of the site extending below the Young Bay Mud layer. Pre-cast concrete piles would be driven from the ground surface to 60 feet deep. Four rows of evenly spaced pre-cast concrete piles would be driven using an impact hammer. The pre-cast concrete piles may also be driven using a vibratory hammer, but this is generally not recommended for deep piles due to constructability concerns. The final arrangement forms a physical buttress.

Alternative 10: Wick Drains with Surcharge and Sheet Piles

This alternative is a combination of a soil improvement and physical buttress method. As described in Alternative 1, using wick drains with surcharge would accelerate consolidation of the Young Bay Mud layer. In this remedial alternative, steel sheet piles are installed along the shoreline perimeter. Sheet piles consist of interlocking steel members driven deep into the soil to form a containment system. The function of the sheet piles is mainly to contain any waste material that could be discharged during a seismic event. Unlike a soil cement gravity wall, the sheet piles are generally weak in bending and would have to be driven deep into the Merritt Sand layer for stability.

Alternative 11: Excavation along Shoreline and Soil Backfill

This alternative consists of excavating the top fill material, the Young Bay Mud layer and any waste material within a narrow zone along the shoreline perimeter extending upland. The width of the excavation would depend on previously predicted failure surfaces (FWENC, 2002). The excavated material would be replaced with soil backfill, which would act as an improved soil zone that would decrease lateral displacements during a seismic event.

Alternative 12: Partial In Situ Solidification

This soil improvement alternative is similar to Alternative 5, where cement slurry is injected and mixed with the Young Bay Mud layer forming solidified columns. The soil mixing is performed using a 5-foot-diameter auger system. This alternative is called partial in situ solidification since only part of the Young Bay Mud layer would be solidified. Rather than a continuous zone along the shoreline, the solidified columns are spaced 10 feet apart on centers. The proposed width of the partial solidified material is 30 feet and extends to a depth of 5 feet into the Merritt Sand layer.

Alternative 13: Soil Bentonite Cutoff Wall

This alternative is a physical buttress method where a cutoff wall is constructed along the shoreline. The cutoff wall is constructed by excavating a trench 3 feet wide and extending 5 feet into the Merritt Sand layer along the shoreline perimeter of the site. The wall consists of a mixture of bentonite clay, imported soil, and water. Imported soil consisting of a silty sand material would be required to ensure the workability of the soil bentonite mixture. The use of cohesive soil sediments from the Young Bay Mud is not recommended because of workability issues.

Once a section of the trench is excavated, bentonite slurry composed of a bentonite clay and water mixture is pumped into the trench. The high density of the slurry mixture would prevent the trench from collapsing. A soil-bentonite mixture prepared by mixing imported soil with bentonite slurry is then used to backfill the excavated trench. The bentonite slurry would be displaced once the trench is backfilled. This process of trench excavation with bentonite slurry and backfilling with soil-bentonite mixture is repeated until the cutoff wall is constructed. Excavated material from the trench would be placed in the landfill area and capped with 2 feet of import fill. Since the excavated material may be contaminated, it would be temporarily capped with the 2-foot-thick import fill layer. This temporary cap may be incorporated into the future 4-foot-thick landfill cap. If determined inadequate for a permanent landfill cap, this 2-foot-thick import fill layer would be replaced with the future 4-foot-thick landfill cap.

Alternative 14: Riprap Embankment in the Bay and Soil Backfill

This alternative is a partial soil improvement method. A proposed riprap embankment is to be constructed along the perimeter shoreline in the water. The riprap embankment would be sloped at least 2 to 1 (horizontal to vertical) in elevation and constructed with a top width of approximately 20 feet. Soil backfill would be placed in the upland area behind the riprap embankment. The Young Bay Mud layer along the slopes would be partially consolidated by placement of soil backfill to increase its shear strength.

Alternative 15: Inclined Timber Piles

This alternative is a physical buttress method and involves installation of 1-foot-diameter timber piles along the shoreline perimeter. The timber piles would be driven at an angle with impact hammers through the toe of the perimeter slopes. The piles would be spaced 3 feet apart and extend 5 feet into the Merritt Sand layer.

Alternative 16: Consolidation with Surcharge

This alternative is a soil improvement method, which involves consolidating the Young Bay Mud layer with surcharge. The fill surcharge would consist of approximately 18 feet of clean soil.

Alternative 17: Wick Drains with Vacuum

This soil improvement alternative involves the use of wick drains and vacuum to accelerate the consolidation of the Young Bay Mud layer.

Instead of applying surcharge to consolidate the Young Bay Mud layer, as proposed in Alternative 1, vacuum is used to remove water from the wick drains. A series of wick drains would be connected with a piping system along the shoreline perimeter extending into the Young Bay Mud layer.

Alternative 18: Vibrated Beam Cement Bentonite Cutoff Wall

This alternative is a type of physical buttress, which involves installation of a cement bentonite cutoff wall around the shoreline perimeter. The wall is constructed by driving a hollow steel beam 5 feet into the Merritt Sand layer using a vibratory hammer. The standard dimensions of the steel beam are 4 inches thick, 3 feet wide, and up to 100 feet in length. When the steel beam reaches the specified depth, a slurry consisting of a mixture of cement, bentonite clay, and water is injected through a series of nozzles connected to the bottom of the steel beam. Injection of slurry continues until the steel beam is fully withdrawn to the ground surface. The steel beam is then driven again and the process continues until the cutoff wall is constructed. Continuity of the cutoff wall is maintained by overlapping each section of the wall. Note that only one steel beam is required for the entire process of constructing the cutoff wall.

The cement bentonite slurry is prepared in a mixing tank and tested for density and viscosity prior to utilization. Its compressive strength varies depending on the mix design, but can generally reach compressive strengths of about 50 pounds per square inch (psi).

Alternative 19: Vibrated Beam Impermix Cutoff Wall

This alternative is the same as Alternative 18, except for the type of slurry used. In this alternative, an Impermix slurry developed by Liquid Earth Support, Incorporated, is used. It consists of a proprietary mixture of attapulgite clay, slag cement, and water. The mixture is prepared in a mixing tank and tested for density and viscosity prior to utilization. The compressive strength of the Impermix slurry varies depending on the mix design, but can normally reach 300 psi.

Alternative 20: Soil Cement Gravity Wall

This alternative is partially a soil improvement method, which results in the creation of a physical buttress. Large-diameter augers are used to inject cement slurry on narrow zones along the shoreline perimeter of the site extending down to the bottom of the Young Bay Mud layer. The cement slurry mixes with existing soil material forming stabilized blocks or columns of soil. The resulting interlocking barrier is commonly described as a soil cement gravity wall.

3.2 EVALUATION CRITERIA

The Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) established the following statutory requirements for the selection of remedial alternatives. The alternatives must:

- Protect human health and the environment.
- Comply with applicable or relevant and appropriate requirements (ARARs), unless a waiver is justified.
- Be cost-effective.
- Utilize permanent solutions and alternative treatment technologies or resource recovery technologies to the maximum extent practicable.
- Satisfy a preference for treatment as a principal element, or provide an explanation in the Record of Decision (ROD) for why the preference was not met.

Nine evaluation criteria are generated based on the above statutory requirements for CERCLA sites. These include: 1) overall protection of human health; 2) compliance with ARARs; 3) long-term effectiveness and permanence; 4) reduction of toxicity, mobility, and volume through treatment; 5) short-term effectiveness; 6) implementability; 7) cost; 8) state or support agency acceptance; and 9) community acceptance. A discussion of each criterion is provided in the following sections. The seismic/geotechnical evaluations performed directly address the CERCLA requirements pertaining to implementability evaluation criteria. However, these evaluations also impact other criteria as well.

3.2.1 Overall Protection of Human Health

This criterion is an overall check of other evaluation criteria such as short-term and long-term effectiveness and compliance with ARARs. It determines whether the specific remedial alternative addresses all potential hazards associated with the site. For this Geotechnical FS Report, the evaluation of overall protection of human health is limited to addressing the geotechnical and seismic hazards identified in the RI Report Addendum, Volume III (FWENC, 2002).

3.2.2 Compliance with ARARs

This evaluation criterion involves identification and compliance with federal and state ARARs. ARARs identified for the geotechnical and seismic evaluation performed are documented in the RI Report Addendum, Volume III (FWENC, 2002). For this Geotechnical FS Report, ARARs are identified for each remedial alternative considered. A compliance check should then be performed to determine if the alternatives meet those requirements. Table 3-2 lists ARARs and the remedial alternatives to which they apply.

Within the 20 remedial alternatives developed for initial screening using the evaluation criteria, there are 12 distinct actions or combinations of actions for which regulatory requirements were evaluated. These actions include: wick drains, surcharge, stone columns, soil/cement gravity wall, bentonite/cement cutoff wall, drilled concrete piers, excavation along shoreline and soil backfill, backfilling, riprap placement, in situ solidification, inclined timber piles, and sheet pile installation. Due to the similarity in installation techniques and applicability of regulatory requirements, the 12 distinct actions were grouped together into seven categories for evaluation in this section. The categories consist of the following:

1. Wick Drains
2. Surcharge
3. Stone Columns
4. Piling (concrete, steel sheet, timber)
5. Riprap
6. Excavation (along shoreline and soil backfill)
7. Soil Cement (gravity wall, bentonite cutoff wall, and in situ solidification)

Regulatory requirements for each of these categories are presented in Sections 3.2.3, 3.2.4, 3.2.5, 3.2.6, 3.2.7, 3.2.8, and 3.2.9. The following discussion refers to the need for certain permits, requirements, and notifications necessary to implement the specific actions. However, since implementation of these activities would be performed under the Navy Installation Restoration (IR) Program and pursuant to CERCLA authority, only the substantive aspects and conditions of these permits and requirements need to be conducted. Specifically, CERCLA 121(e)(1), 42 United States Code (USC), Section 9621(e)(1), states that "No Federal, State, or local permit shall be required for the portion of any removal or remedial action conducted entirely on site, where such remedial action is selected and carried out in compliance with this section."

TABLE 3-2

APPLICABLE OR RELEVANT AND APPROPRIATE REQUIREMENTS (ARARS)

ARAR No.	ARAR Code Citation	Description	Applicability
1	Alameda City Ordinance 13-56	Drilling and/or monitoring well permits from the City of Alameda are required. While the wick drains are not intended to be groundwater wells, they will exhibit groundwater well characteristics once installed, as they are intended to collect water from the surrounding formation.	Wick Drains Stone Column
2	RWQCB Basin Plan 1995	Pursuant to Chapter 2 (beneficial uses) and Chapter 3 (water quality objectives) of the San Francisco RWQCB Basin Plan, soil additives must be evaluated to ensure that they will not present a threat of contamination to the surrounding environment either through downward migration into groundwater or through a potential surface release.	Wick Drains Soil Cement
3	40 CFR, Part 6.302	Pursuant to Executive Order 11990, wetland areas must be identified and delineated to prevent impact to these areas. Where impact is unavoidable, appropriate mitigation measures must be employed and a certification authorizing work in a wetland area must be obtained pursuant to Section 401 of the CWA.	Wick Drains Surcharge Stone Column Piling Riprap Excavation Soil Cement
4	33 USC, Section 1344	A Section 404 permit, under the CWA, from the Army Corp of Engineers, may be required if the installation of the device/materials will constitute or will require dredging or filling within navigable waters or alteration of wetland areas.	Wick Drains Surcharge Stone Column Piling Riprap Excavation Soil Cement
5	16 USC, Section 1451-1464 Title 14 CCR, Sections 13001-13666.4	Work in the area near or along the shoreline may be subject to the Federal and California State Coastal Zone Management Act and must be consistent with the state management programs. The approved state management plan for San Francisco Bay consists of the McAteer-Petris Act and the San Francisco Bay Plan, developed pursuant to the act.	Wick Drains Surcharge Stone Column Piling Riprap Excavation Soil Cement

TABLE 3-2 (Continued)

APPLICABLE OR RELEVANT AND APPROPRIATE REQUIREMENTS (ARARs)

ARAR No.	ARAR Code Citation	Description	Applicability
6	16 USC, Section 470	Pursuant to Executive Order 11593, project activities involving excavation or other land-disturbing activities are subject to review for cultural, archaeological, and historical resources. Performing surveys or referencing surveys previously conducted for the project areas may be required.	Wick Drains Surcharge Stone Column Piling Riprap Excavation Soil Cement
7	16 USC, Section 1536(a), (h), (i), (b) California Fish & Game Code Section 2080	In accordance with the Federal and California State Endangered Species Act, the U.S. Fish and Wildlife Service and the California Department of Fish and Game may need to be consulted, and a determination of the presence and potential impact to endangered species or habitat may need to be assessed.	Wick Drains Surcharge Stone Column Piling Riprap Excavation Soil Cement
8	Title 40 CFR, Parts 122-124	Title 40 CFR, Parts 122, 123, and 124, contain requirements to control stormwater discharges associated with construction activities exceeding 5 acres in size.	Wick Drains Surcharge Stone Column Piling Riprap Excavation Soil Cement
9	Title 27 CCR, Sections 20385, 20420, and 20425	Title 27 CCR, Section 20385, Section 20420, and Section 20425 establish groundwater monitoring program requirements for waste management units. These requirements include a detection monitoring program to determine effectiveness of the selected remedy, and an evaluation program to assess the nature and extent of a release, if discovered.	Surcharge Soil Cement
10	Title 27 CCR, Sections 20080(b), (c) and 21090.	Title 27 CCR landfill requirements may be applicable to the landfill cap to be deployed on the landfill.	Surcharge
11	Title 22 CCR, Section 6626124	Pursuant to the California RCRA program for hazardous waste management, 22 CCR, Section 66261.24 requires waste to be characterized for appropriate disposal.	Excavation Soil Cement

Notes:

CCR - California Code of Regulations
CFR - Code of Federal Regulations
CWA - Clean Water Act
RCRA - Resource Conservation and Recovery Act
RWQCB - Regional Water Quality Control Board
USC - United States Code

3.2.3 Long-Term Effectiveness and Permanence

The evaluation of long-term effectiveness and permanence of the remedial alternative refers to potential risks remaining after the alternative has been implemented. Future operation and maintenance (O&M) issues should be addressed in this evaluation.

3.2.4 Reduction of Toxicity, Mobility, and Volume Through Treatment

This evaluation criterion directly addresses the statutory requirement for reduction of toxicity, mobility, and volume of waste through treatment. The site contains waste from landfill operations. However, treatment of media such as air, soil, or groundwater is not addressed in this Geotechnical FS Report.

For this Geotechnical FS Report, the primary concern is waste release into San Francisco Bay from static or seismic instability. Other concerns include discharge of impacted soil/water to the ground surface and lateral spreading of the current waste across the site. The remedial alternatives considered will be evaluated based on the above concerns.

3.2.5 Short-Term Effectiveness

Short-term effectiveness of a remedial alternative refers to the performance of the alternative during the construction or implementation phase. Issues that should be evaluated include protection of community and workers during remedial actions, environmental impacts, and the duration for completion of the remedial alternative implementation.

3.2.6 Implementability

Implementability evaluation involves technical and administrative feasibility. Technical feasibility refers to meeting the established performance criteria, constructability, quality control requirements during construction, long-term maintenance, and additional future remedial actions associated with the alternative. Administrative feasibility refers to the ability to obtain approvals from applicable agencies and the availability of specific equipment and technical specialists.

3.2.7 Cost

The cost evaluation consists of estimating the capital costs and O&M costs. Capital costs include both direct (construction) and indirect (nonconstruction and overhead) costs. O&M costs are post-construction costs related to the remedial alternative. The level of accuracy of a remedial alternative cost estimate should be +50 percent to -30 percent of the eventual actual cost. A present worth analysis should be conducted to convert all costs associated to a single base year (normally the current year). This will allow comparisons to be made between remedial alternatives with different construction duration and O&M costs. A discount rate of 3.9 percent before taxes and after inflation can be used and was recommended in the U.S. Environmental

Protection Agency (EPA) document entitled *The Role of Cost in the Superfund Remedy Selection Process* (EPA, 1996). Also, for cost estimate purposes, the period of performance for calculating O&M costs was assumed to be for a period of up to 30 years. For certain alternatives, a sensitivity analysis might be performed to refine the design.

3.2.8 State or Support Agency Acceptance

This evaluation criterion deals with the concerns of the state or support agency regarding technical and administrative issues. The preferred remedial alternative will be presented to the public in a Proposed Plan (PP). This allows the public and government regulators a reasonable opportunity to comment on the preferred remedial alternative. After comments on the Remedial Investigation/Feasibility Study (RI/FS) Report and PP are received, the lead regulatory agency prepares a responsiveness summary that documents the final remedial alternative and addresses the acceptance criteria in a ROD.

3.2.9 Community Acceptance

Community acceptance must be evaluated with regard to issues and concerns the public may have regarding each remedial alternative. This criterion will be evaluated in the ROD once comments on the RI/FS Report and PP have been received.

3.3 SCREENING OF ALTERNATIVES

An initial screening evaluation was performed based on the following three factors: effectiveness, implementability, and cost. The effectiveness evaluation is associated with the first five evaluation criteria discussed in Section 3.2, which include: 1) overall protection of human health and environment; 2) compliance with ARARs; 3) long-term effectiveness and permanence; 4) reduction in toxicity, mobility, and volume through treatment; and 5) short-term effectiveness. Implementability is based on technical and administrative feasibility. Technical feasibility refers to meeting the established performance criteria, constructability, quality control issues during construction, long-term maintenance, and additional future remedial actions associated with the remedial alternative. Administrative feasibility refers to the ability to obtain approvals from (applicable) agencies and availability of specific equipment and technical specialists. The cost evaluation is based on relative cost-effectiveness among remedial alternatives since no cost estimate has been developed at this initial stage of screening.

Table 3-3 summarizes the initial screening evaluation. The table shows each remedial alternative evaluated based on effectiveness, implementability, and cost. A decision on whether a remedial alternative was selected for more detailed analyses is included in the table. Nine remedial alternatives (Alternatives 1 to 9) as shown in Table 3-3 were selected. Alternatives 10 to 20 were not selected and were not evaluated further.

TABLE 3-3

INITIAL SCREENING OF ALTERNATIVES

Alternative No.	Description	Primary Hazard Addressed	Effectiveness	Implementability	Cost	Screening Comments and Decision
1	Wick Drains with Surcharge	Slope Instability	• No impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Possibility of collecting impacted water in wick drains. Controlled release of water at the surface is a minor concern. • Acceptable short-term effectiveness.	• Moderate potential for meeting performance criteria. No constructability concerns. Time is a factor since the Young Bay Mud layer will take years to consolidate. • Acceptable administrative feasibility.	• Low capital • Moderate O&M	SELECTED (for detailed analysis) Low cost and feasible to implement.
2	Stone Columns with Surcharge	Liquefaction/ Lateral Spreading	• No impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Possibility of collecting impacted water in stone columns. Controlled release of water and soil cuttings at the surface is a minor concern. Involves potential removal of impacted soil. The removal could generate a health hazard during the excavation, handling, and disposal activities. • Acceptable short-term effectiveness.	• Moderate potential for meeting performance criteria. No constructability concerns. Time is a factor since the Young Bay Mud layer will take years to consolidate. • Acceptable administrative feasibility.	• Moderate capital • Moderate O&M	SELECTED (for detailed analysis) Feasible to implement.
3	Sheet Piles with Anchors.	Slope Instability	• No impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness with minor concern of degradation of piles. • Involves potential removal of impacted soil. The removal could generate a health hazard during the excavation, handling, and disposal activities. • Acceptable short-term effectiveness.	• Moderate potential for meeting performance criteria. Degradation of steel sheet pile is a minor concern since it is in contact with water/soil. Some maintenance/monitoring is needed. • Acceptable administrative feasibility.	• Low capital • Low O&M	SELECTED (for detailed analysis) Low cost and feasible to implement.
4	Stone Columns with Surcharge and Sheet Piles	Both Liquefaction and Slope Instability	• No impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness with minor concern of degradation of piles. • Possibility of collecting impacted water in stone columns. Controlled release of water and soil cuttings at the surface is a minor concern. Involves potential removal of impacted soil. The removal could generate a health hazard during the excavation, handling, and disposal activities. • Acceptable short-term effectiveness.	• High potential for meeting performance criteria. Degradation of steel sheet pile is a minor concern since it is in contact with water/soil. Some maintenance/monitoring is needed. Time is a factor since the Young Bay Mud layer will take years to consolidate. • Acceptable administrative feasibility.	• High capital • Moderate O&M	SELECTED (for detailed analysis) High cost, but technically feasible.

TABLE 3-3 (Continued)

INITIAL SCREENING OF ALTERNATIVES

Alternative No.	Description	Primary Hazard Addressed	Effectiveness	Implementability	Cost	Screening Comments and Decision
5	Soil Cement Gravity Wall and Stone Columns	Both Liquefaction and Slope Instability	• Very low impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Cement slurry is mixed with soil, but is not a hazard. Controlled release of water and soil cuttings at the surface is a minor concern. Involves potential removal of impacted soil. The removal could generate a health hazard during the excavation, handling, and disposal activities. • Acceptable short-term effectiveness.	• High potential for meeting performance criteria. Some maintenance/monitoring is needed. Time is not a major factor since the stone columns are placed in the fill layer, which takes a much shorter time to consolidate compared to the Young Bay Mud layer. • Acceptable administrative feasibility.	• Moderate capital • Moderate O&M	SELECTED (for detailed analysis) Feasible to implement.
6	Concrete Wall	Both Liquefaction and Slope Instability	• Very low impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Involves potential removal of large volume of impacted soil. The removal could generate a health hazard during the excavation, handling, and disposal activities. • Acceptable short-term effectiveness.	• High potential for meeting performance criteria. • Acceptable administrative feasibility.	• Moderate capital • Low O&M	SELECTED (for detailed analysis) Feasible to implement.
7	Excavation with Riprap	Both Liquefaction and Slope Instability	• Moderate impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Involves potential removal of impacted soil. The removal could generate a health hazard during the excavation, handling, and disposal activities. Also, riprap will have to be dumped into San Francisco Bay. • Short-term stability concerns during construction.	• High potential for meeting performance criteria. However, there is a slope stability concern during excavation and prior to riprap placement. • Acceptable administrative feasibility. However, more difficult to obtain regulatory approvals due to removal of potential waste and placement of riprap in San Francisco Bay.	• Moderate capital • Low O&M	SELECTED (for detailed analysis) Feasible to implement even with several concerns.
8	Drilled Concrete Piers with Stone Columns	Both Liquefaction and Slope Instability	• No impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Controlled release of potentially impacted soil cuttings at the surface is a minor concern. • Acceptable short-term effectiveness.	• High potential for meeting performance criteria. • Acceptable administrative feasibility.	• Moderate capital • Low O&M	SELECTED (for detailed analysis) Feasible to implement.

TABLE 3-3 (Continued)

INITIAL SCREENING OF ALTERNATIVES

Alternative No.	Description	Primary Hazard Addressed	Effectiveness	Implementability	Cost	Screening Comments and Decision
9	Pre-cast Concrete Piles	Slope Instability	• No impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • No surface release of impacted water/soil. • Acceptable short-term effectiveness.	• High potential for meeting performance criteria. • Acceptable administrative feasibility.		SELECTED (for detailed analysis) Feasible to implement and not many ARARs.
10	Wick Drains with Surcharge and Sheet Piles	Both Liquefaction and Slope Instability	• No impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness with minor concern of degradation of piles. • Possibility of collecting impacted water in wick drains and release at surface. • Acceptable short-term effectiveness.	• Moderate potential for meeting performance criteria. Degradation of steel sheet pile is a minor concern since it is in contact with water/soil. Some maintenance/monitoring is needed. Time is a factor since the Young Bay Mud layer will take years to consolidate. • Acceptable administrative feasibility.	• Very high capital • Very high O&M	NOT SELECTED (no further evaluation) Relatively very high cost.
11	Excavation along Shoreline and Soil Backfill	Both Liquefaction and Slope Instability	• Moderate impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Involves removal of impacted soil. • The removal will generate a health hazard during the excavation, handling, and disposal activities.	• Remedial action objective is satisfied since waste will be removed; however, there is a slope stability concern during excavation. • Acceptable administrative feasibility. However, more difficult to obtain regulatory approvals due to removal of waste.	• Very high capital • Low O&M	NOT SELECTED (no further action) Significant waste handling and disposal cost.
12	Partial In Situ Solidification	Both Liquefaction and Slope Instability	• Very low impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Cement slurry will mix with soil, but is not a hazard. Controlled release of water and soil cuttings at the surface is a minor concern. • Acceptable short-term effectiveness.	• Low potential for meeting performance criteria since only a partial solidification is performed (compared to Alternative 5). • Acceptable administrative feasibility.	• Moderate capital • Low O&M	NOT SELECTED (no further evaluation) Technically not feasible.

TABLE 3-3 (Continued)

INITIAL SCREENING OF ALTERNATIVES

Alternative No.	Description	Primary Hazard Addressed	Effectiveness	Implementability	Cost	Screening Comments and Decision
13	Soil Bentonite Cutoff Wall	Both Liquefaction and Slope Instability	• Very low impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Bentonite will mix with soil, but is not considered a hazard. Controlled release of soil cuttings at the surface is a minor concern. • Acceptable short-term effectiveness.	• Low potential for meeting performance criteria because the shear strength of the soil-bentonite mixture is generally low. • Acceptable administrative feasibility.	• Low capital • Low O&M	NOT SELECTED (no further evaluation) Technically not feasible.
14	Riprap Embankment in the Bay and Soil Backfill	Slope Instability	• Moderate impact to human health and environment. • Complies with ARARs. • Long-term effectiveness is low due to questionable stability of riprap embankments. • Waste movements contained during construction. • Acceptable short-term effectiveness.	• Low potential for meeting performance criteria because the stability of embankments during a seismic event is questionable. Also, partial consolidation of the Young Bay Mud layer normally takes several years. • More difficult to obtain regulatory approvals due to material placement in San Francisco Bay	• Low capital • Low O&M	NOT SELECTED (no further evaluation) Technically not feasible.
15	Inclined Timber Piles	Slope Instability	• Low impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness with minor concern of degradation of piles. • No surface release of impacted water/soil. • Acceptable short-term effectiveness.	• Low potential for meeting performance criteria. Degradation of timber pile is a concern since it is in contact with water/soil. Also constuctability and quality control issues with maintaining pile alignment. Maintenance/monitoring is needed. • Acceptable administrative feasibility.	• Low capital • Low O&M	NOT SELECTED (no further evaluation) Technically not feasible.
16	Consolidation with Surcharge	Slope Instability	• Moderate impact to human health and environment. • Complies with ARARs. • Long-term effectiveness is questionable due to significant fill material required. • Waste materials are contained. • Short-term effectiveness is questionable due to significant fill material required.	• Low potential for meeting performance criteria since significant fill material required for consolidation. The length of time for consolidation of the Young Bay Mud layer is also an issue. • Riprap placement in the San Francisco Bay makes it more difficult for regulatory approval.	• Low capital • Moderate O&M	NOT SELECTED (no further evaluation) Very high cost and constructability concerns regarding placement of rip rap.

TABLE 3-3 (Continued)

INITIAL SCREENING OF ALTERNATIVES

Alternative No.	Description	Primary Hazard Addressed	Effectiveness	Implementability	Cost	Screening Comments and Decision
17	Wick Drains with Application of Vacuum	Slope Instability	• No impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Possibility of collecting impacted water in wick drains. Controlled release of water at the surface is a minor concern. • Acceptable short-term effectiveness.	• Low potential for meeting performance criteria due to questionable effectiveness of vacuum system. Also, consolidation of the Young Bay Mud generally takes years. • Acceptable administrative feasibility.	• Moderate capital • Moderate O&M	NOT SELECTED (no further evaluation Technically not feasible.
18	Vibrated Beam Cement Bentonite Cutoff Wall	Both Liquefaction and Slope Instability	• Very low impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Slurry will be pumped into the ground, but is not considered a hazard. Controlled release of soil cuttings at the surface is a minor concern. • Acceptable short-term effectiveness.	• Very low potential for meeting performance criteria because the compressive strength of the mixture is low. In addition, there is concern about continuity of the cutoff wall related to constructability and quality control. • Acceptable administrative feasibility.	• Low capital • Low O&M	NOT SELECTED (no further evaluation) Technically not feasible. Constructability concerns.
19	Vibrated Beam Impermix Cutoff Wall	Both Liquefaction and Slope Instability	• Very low impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Slurry will be pumped into the ground, but is not considered a hazard. Controlled release of soil cuttings at the surface is a minor concern. • Acceptable short-term effectiveness.	• Very low potential for meeting performance criteria because the compressive strength of the mixture is low (higher than in Alternative 18, but generally still low). In addition, there is concern about continuity of the cutoff wall related to constructability and quality control. • Acceptable administrative feasibility.	• Low capital • Low O&M	NOT SELECTED (no further evaluation) Technically not feasible. Constructability concerns.
20	Soil Cement Gravity Wall	Both Liquefaction and Slope Instability	• Very low impact to human health and environment. • Complies with ARARs. • Acceptable long-term effectiveness. • Cement slurry is mixed with soil, but is not considered a hazard. Controlled release of water and soil cuttings at the surface is a minor concern. • Acceptable short-term effectiveness.	• High potential for meeting performance criteria. Performance similar to Alternative 5 but would cost more because a larger dimension gravity wall would be required as compared to Alternative 5. • Acceptable administrative feasibility.	• High capital • Moderate O&M	NOT SELECTED (no further evaluation) High cost.

Notes:

ARAR – applicable or relevant and appropriate requirement
O&M – operation and maintenance

4.0 DETAILED ANALYSIS OF SELECTED ALTERNATIVES

This section includes detailed analysis of the nine remedial alternatives selected based on initial screening performed in the previous section. The detailed analysis involves further screening of these selected alternatives based on the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) evaluation criteria (Section 3.2). The first step of the detailed analysis for further screening involved implementability analysis of these selected alternatives. Cost evaluation of the alternatives that satisfied the implementability criteria was then performed. The remaining alternatives that satisfied both cost and implementability evaluation criteria are then subject to a comparative analysis. The comparative analysis involves evaluation and comparison of the remaining alternatives based on all nine CERCLA evaluation criteria. Based on the findings of the comparative analysis, a recommended remedial alternative was selected to address geotechnical and seismic hazards identified in the Remedial Investigation (RI) Report Addendum, Volume III [Foster Wheeler Environmental Corporation (FWENC), 2002].

4.1 DESCRIPTION OF ALTERNATIVES

Each of the nine selected alternatives is described in detail in the following subsections. The descriptions provide conceptual level details pertaining to the application of each alternative. Technical limitations and design assumptions are discussed when applicable. Methods of construction, including constructibility concerns, if any, are presented.

The selected alternatives improve stability of the site perimeter slopes either by increasing shear strength of the site soils (particularly Young Bay Mud) and/or installing a physical buttress. The perimeter soil slopes combined with the improved soil zone or installed physical buttress act as a retaining structure confining the waste.

4.1.1 Alternative 1 - Wick Drains with Surcharge

This alternative includes the installation of wick drains to accelerate the consolidation of the Young Bay Mud layer and application of surcharge for consolidation. This alternative assumes that the proposed golf course will be constructed after the Young Bay Mud layer has fully consolidated.

Wick Drains

Long-term consolidation of thick compressible soft silts and clays may take 10 to 20 years to complete. The use of wick drains accelerates the consolidation process. Wick drains create closely spaced vertical drainage paths for the pore water pressure to dissipate quickly under the application of a surcharge. The consolidation will take place within a few months. Additional details of the consolidation potential and time period required to fully consolidate the Young Bay Mud layer can be determined during the detailed design phase.

Wick drains would be installed in selected areas of the site affecting the slope failure plane to create closely spaced artificial vertical drainage paths to which the pore water can flow, thus decreasing the consolidation time from years to months. Wick drains consist of a central plastic core, which functions as a free-draining water channel, surrounded by a thin geotextile fabric. A typical wick drain is approximately 4 inches wide, ¼ inch thick, and comes in rolls up to 1,000 feet in length.

Wick drains are installed with stitchers mounted on either backhoes or cranes. The wick drain is hydraulically pushed or vibrated into the ground to the desired depth, typically to the bottom of the soft-soil stratum.

The wick drains would be installed in a narrow area extending from the shoreline to approximately 95 feet upland along the shoreline perimeter as shown in Figure 4-1. The wick drains would be spaced every 5 feet in a rectangular pattern from the ground surface to the bottom of the Young Bay Mud layer. The configuration of the wick drains is shown in Figure 4-2. Varying depths of the wick drains are shown in the cross sections presented in Figures 4-3 through 4-7.

Surcharge

A surcharge consisting of import fill material would be placed to consolidate the Young Bay Mud layer. Thickness and width of the fill surcharge are estimated to be 18 feet and 200 feet, respectively, for full consolidation of the Young Bay Mud layer. The surcharge would be left in place until full consolidation of the Young Bay Mud occurs. The time of consolidation was not determined since this alternative was technically not feasible based on other considerations as indicated in Section 4.2.4.

The fill surcharge would be installed as shown in Figures 4-3 through 4-7.

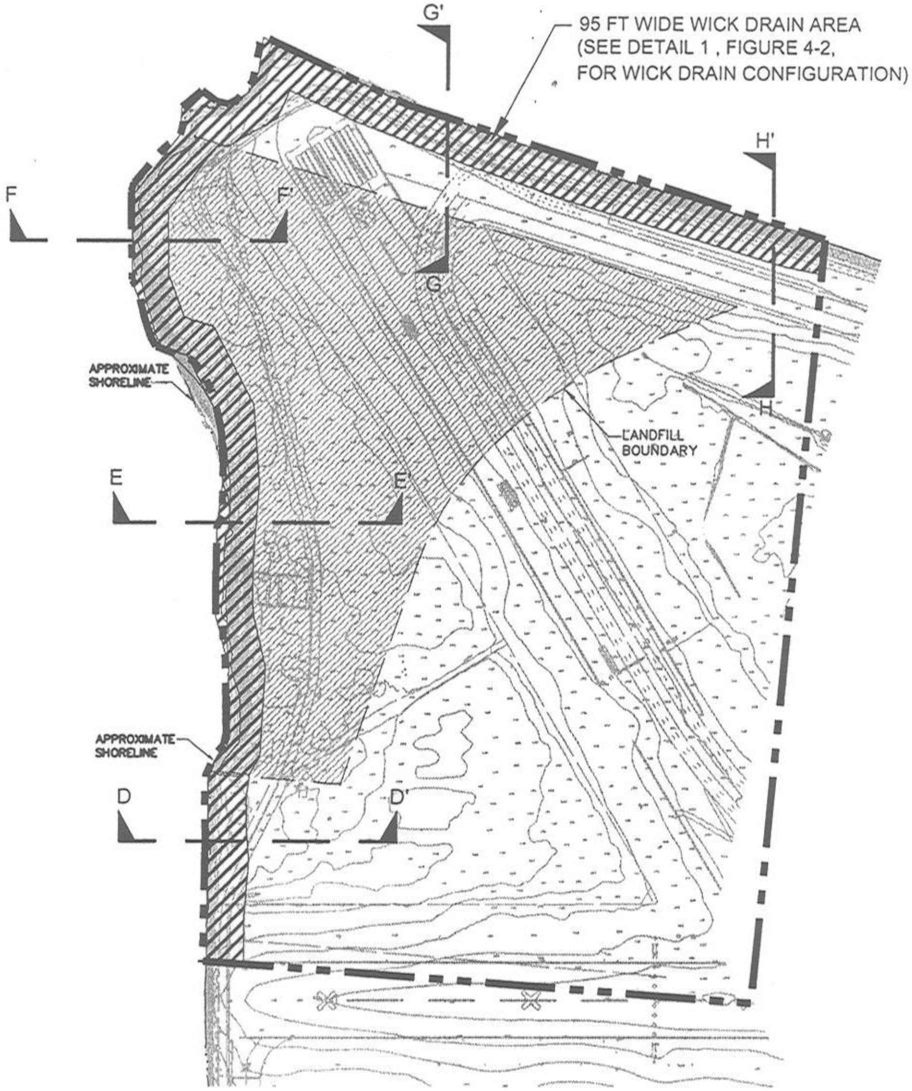
4.1.2 Alternative 2 - Stone Columns with Surcharge

This alternative includes the installation of stone columns across narrow zones along the perimeter extending from the ground surface to the Young Bay Mud layer. A surcharge load is used to consolidate the Young Bay Mud layer, as well as densify the fill layer. This leads to higher shear strengths for the soil layers and reduced liquefaction potential. This alternative assumes that the proposed golf course will be constructed after the Young Bay Mud layer has fully consolidated.

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	03136841.DWG

I:\1990-RAC\CTO-0054\DWG\031368\03136841.DWG
PLOT/UPDATE: SEP 12 2003 15:48:55

REFERENCE:
HJW-GeoSpatial, Inc., Upland Topography
NAD27, NGVD29 – CCS Zone III



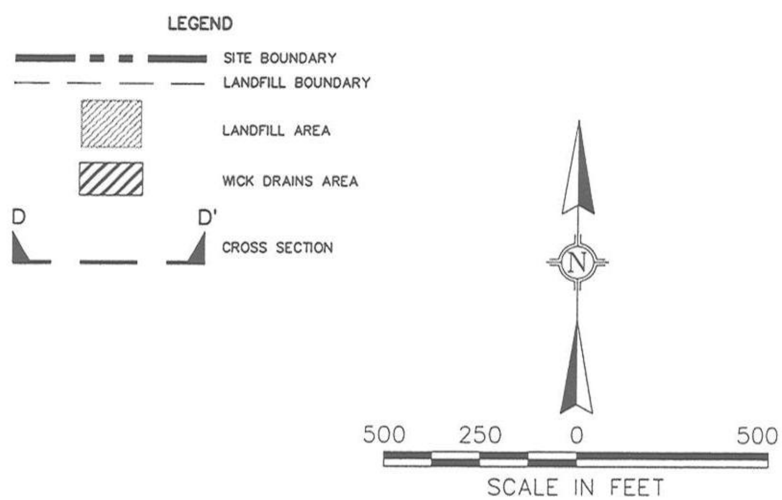
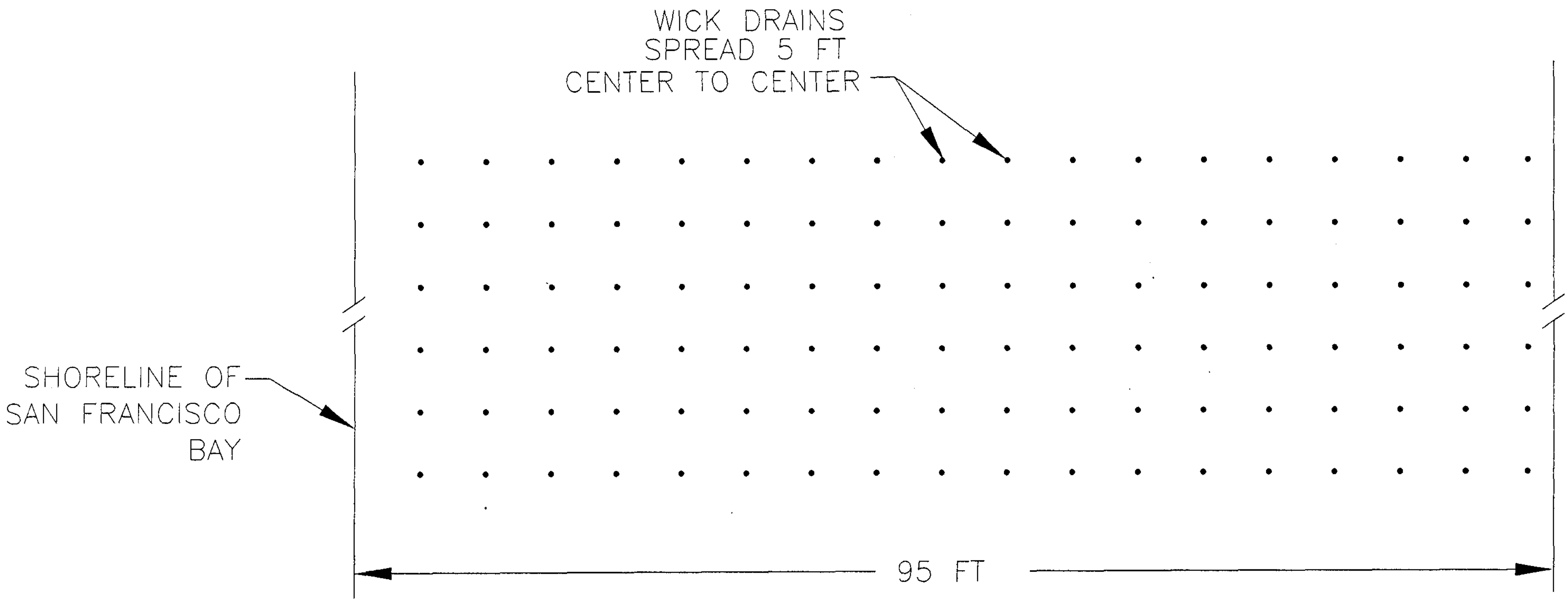


Figure 4-1
ALTERNATIVE 1: WICK DRAINS WITH SURCHARGE (PLAN VIEW)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 03136842.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



NOT TO SCALE

Detail 1 – Wick Drain Configuration

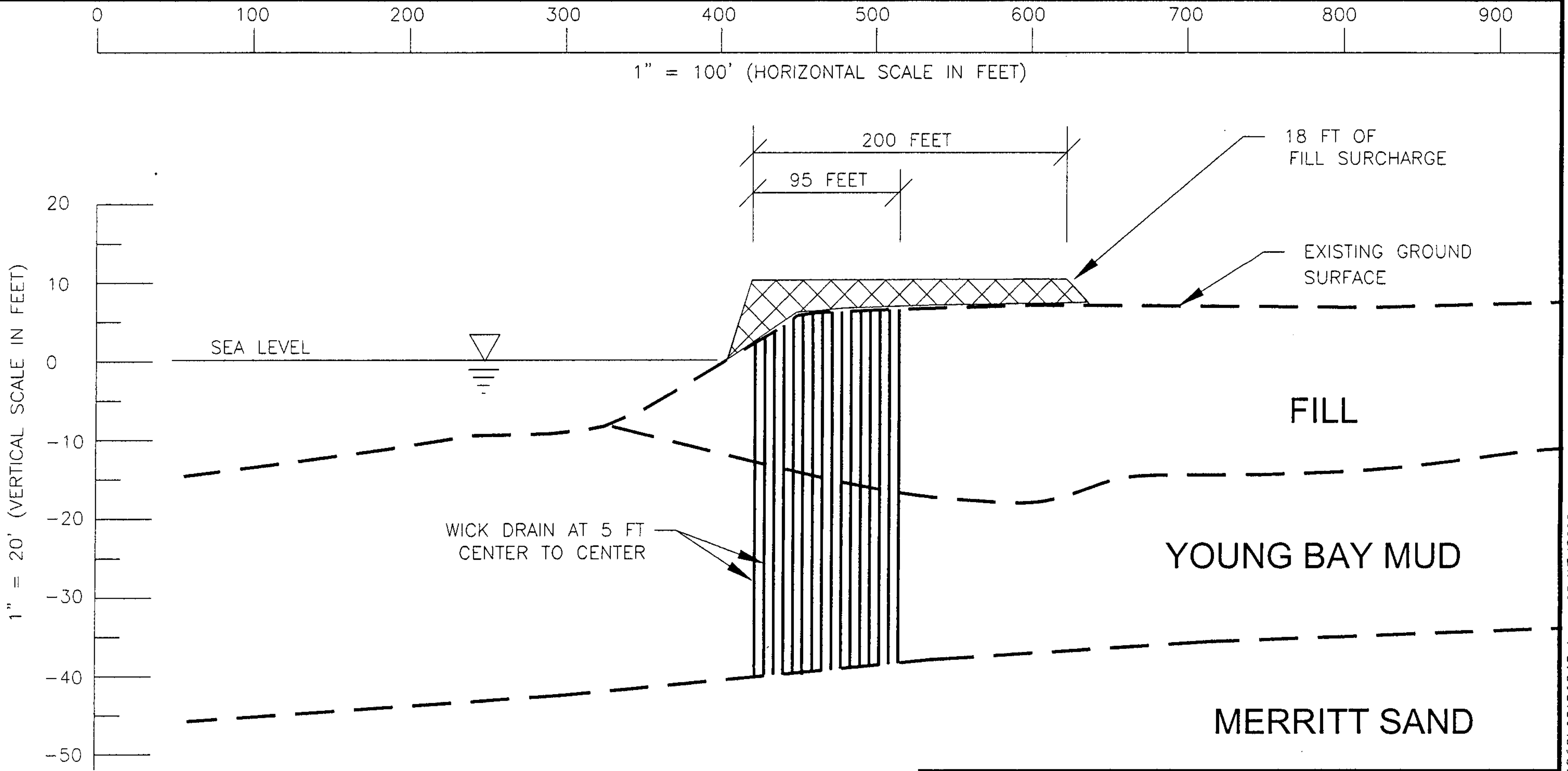
Figure 4–2
ALTERNATIVE 1: WICK DRAINS WITH SURCHARGE (DETAIL 1)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\03136842.dwg 09/12/2003 04:21:40 PM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 03136843.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION D-D' (LOOKING NORTH)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

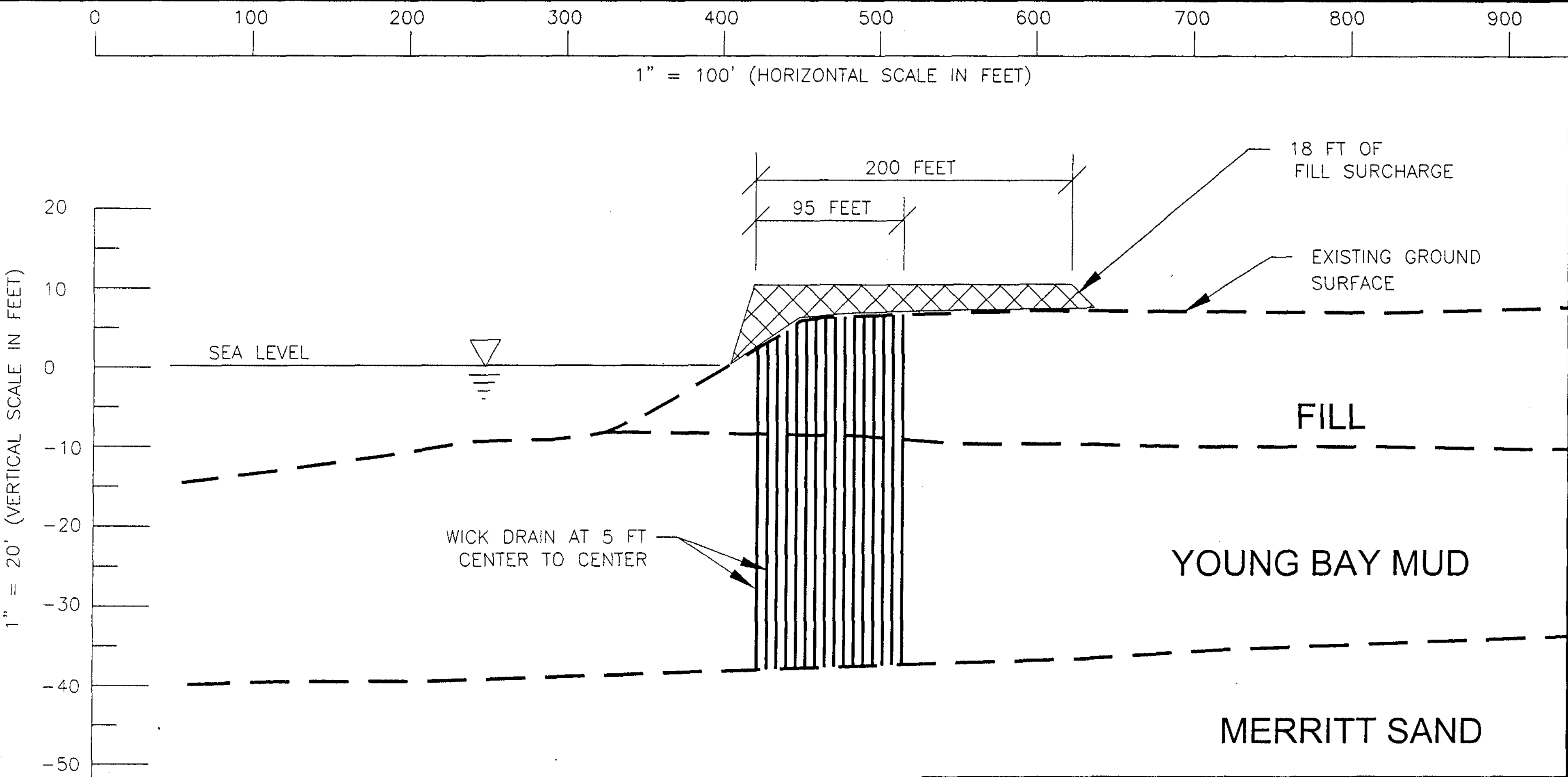
Figure 4-3
ALTERNATIVE 1: WICK DRAINS WITH SURCHARGE (SECTION D-D')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\03136843.dwg 09/15/2003 10:01:31 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 03136844.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION E-E' (LOOKING NORTH)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

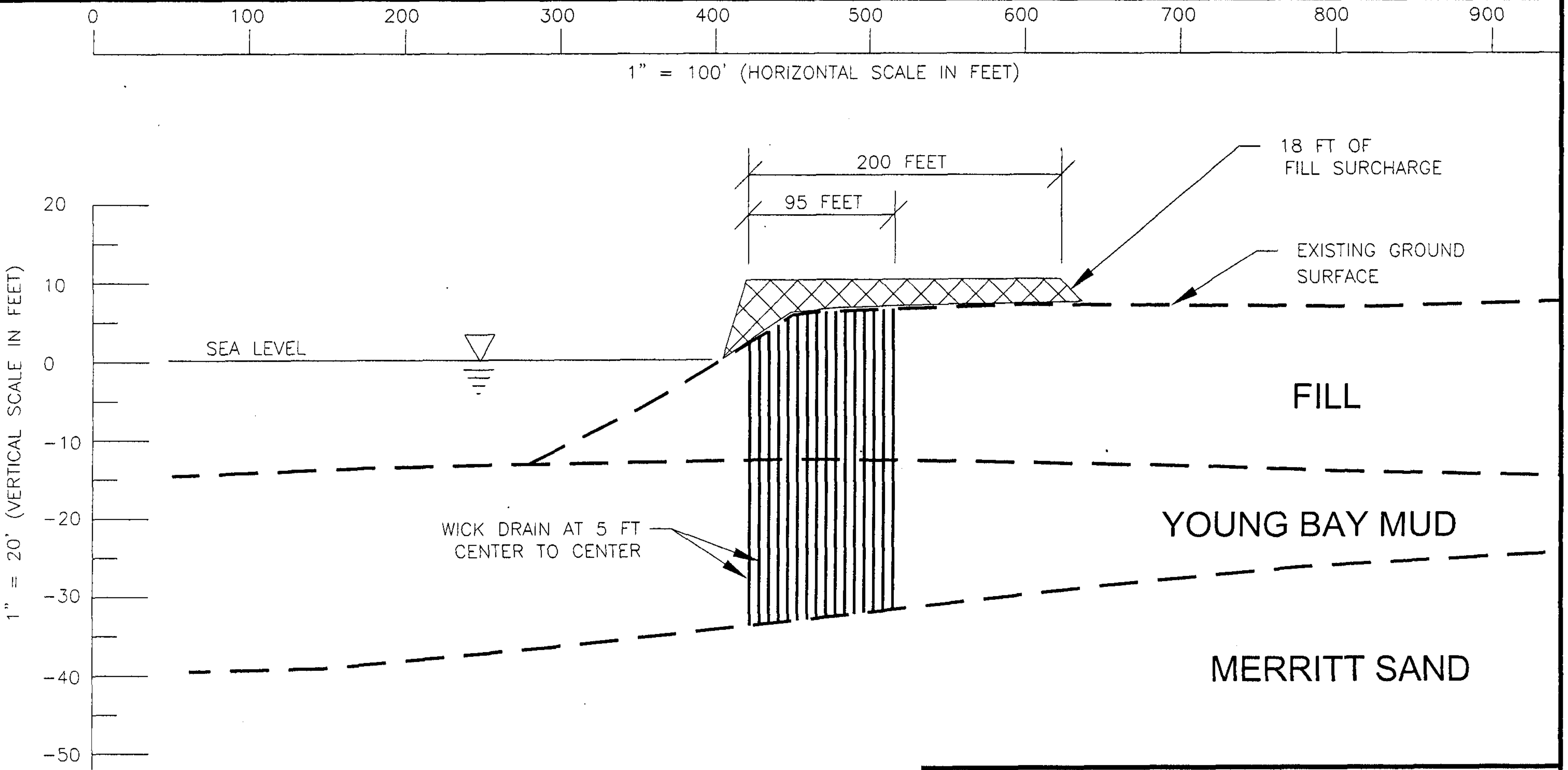
Figure 4-4
ALTERNATIVE 1: WICK DRAINS WITH SURCHARGE (SECTION E-E')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\03136844.dwg 09/15/2003 10:27:01 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 03136845.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION F-F' (LOOKING NORTH)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

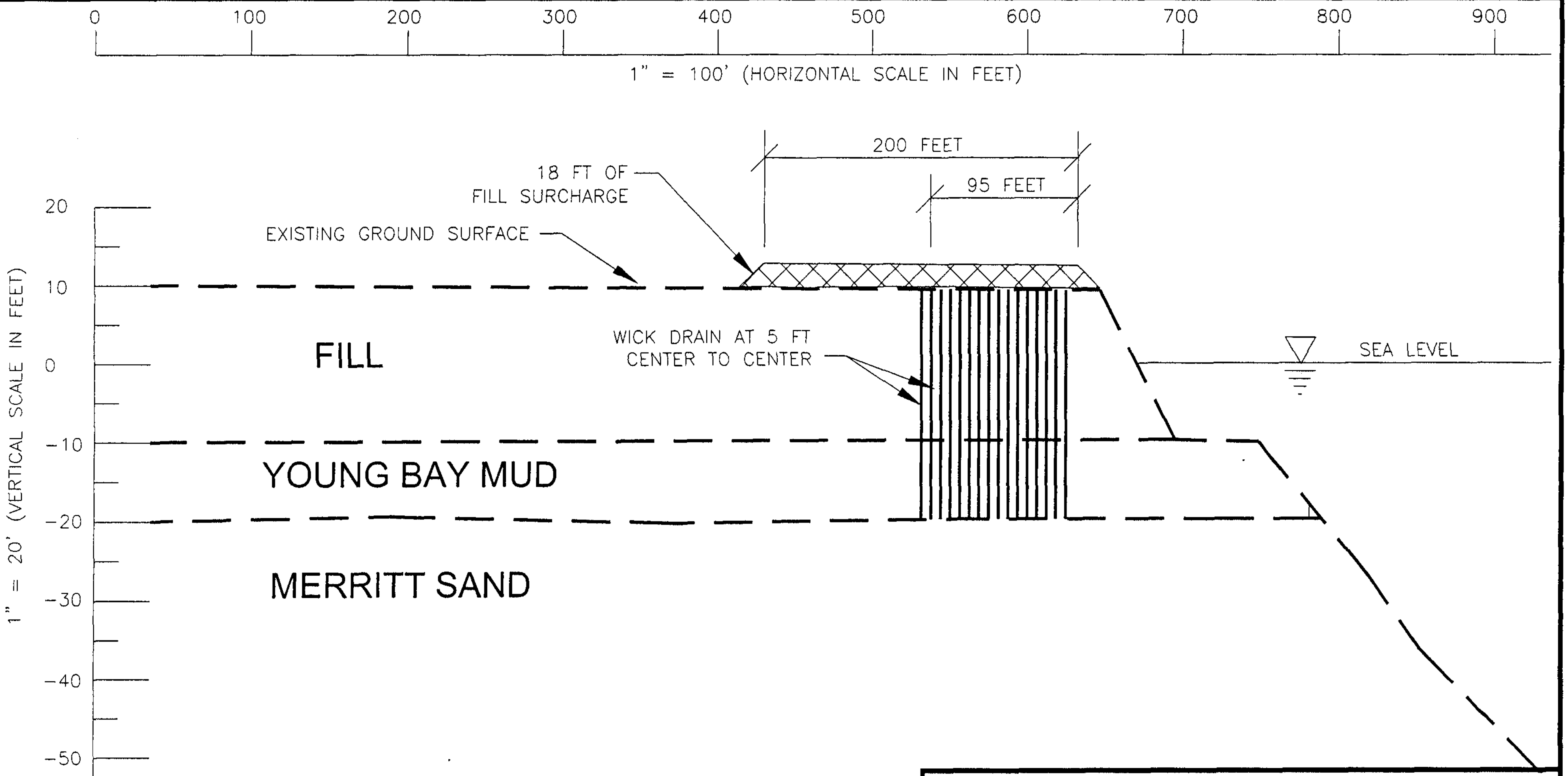
Figure 4-5
ALTERNATIVE 1 - WICK DRAINS WITH SURCHARGE (SECTION F-F')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\03136845.dwg 09/15/2003 10:41:16 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 03136846.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION G-G' (LOOKING WEST)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

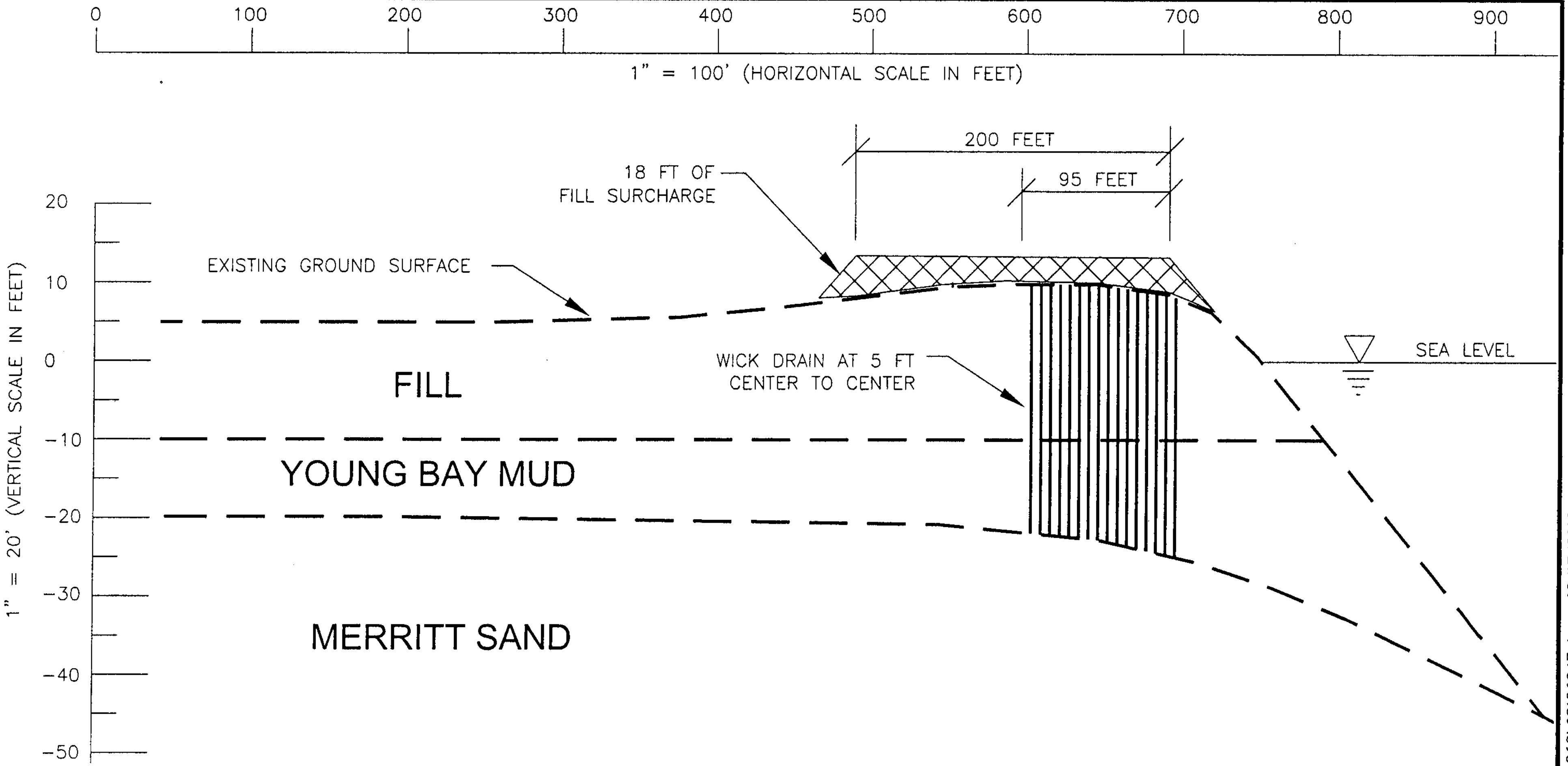
Figure 4-6
ALTERNATIVE 1 - WICK DRAINS WITH SURCHARGE (SECTION G-G')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\03136846.dwg 09/15/2003 11:01:34 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 03136847.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION H–H' (LOOKING WEST)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

Figure 4–7
ALTERNATIVE 1 – WICK DRAINS WITH SURCHARGE (SECTION H–H')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\03136847.dwg 09/15/2003 11:15:40 AM PDT

Stone Columns

Stone columns create vertical drainage paths for existing pore water pressure to dissipate relatively quickly under the application of a surcharge. Stone columns would be installed along the shoreline perimeter of the site extending down to the Young Bay Mud layer. The installation of stone columns would replace existing weak soils (fill material and Young Bay Mud) with higher strength stones (increased shear strength) and densify the surrounding soils. Commonly used methods to install stone columns include the wet top feed, dry bottom feed, and "Frankie" stone column methods as described in Section 3.1 (see initial description of Alternative 2: Stone Columns with Surcharge). The size of the stones will be determined during the design phase.

This soil improvement method would also involve accelerating the process of consolidation in the Young Bay Mud layer. Additionally, in cohesive soils such as Young Bay Mud, the stone backfill in its densified state performs as a structural reinforcement element to increase the bearing capacity of the mass, and it greatly reduces settlements. In the granular soils of the upper fill layer, stone columns are used to enhance drainage and subsequently assist in the densification process, resulting in improvement of lateral stability and reduction of the fill soils lateral. The soil densification and increased drainage also reduce liquefaction potential in the fill layer.

The stone columns would be installed in an area extending from the shoreline to approximately 38 feet into the upland area and along the shoreline perimeter as shown in Figure 4-8. It would consist of 3-foot-diameter columns. Figure 4-9 shows the configuration of the stone columns. The stone columns would extend from the ground surface to the bottom of the Young Bay Mud layer as shown in Figures 4-10 through 4-14.

The densification of surrounding granular soils results in an increase of the soil friction angle and shear strength. To account for the shear strength increase of the upper fill layer soils in the slope stability analyses, the width of the improved zone was artificially divided into two parts. These are:

1. An approximate 20-foot-wide zone representing the improved granular soils surrounding the stone columns, where the soil friction angle was increased from 32 to 34 degrees.
2. An 18-foot-wide zone with a friction angle of 40 degrees, representing the stone column mass inserted in the 38-foot-wide improved soil zone.

The 38-foot-wide improved soil zone will not liquefy and will, therefore, act as a massive stabilizing buttress immediately in front of the liquefiable soils in the upper fill layer. Stability analyses to evaluate post-earthquake stability of the site slopes will use residual shear strength of liquefied soils inboard of the improved fill zone.

Surcharge

A surcharge consisting of import fill material would be placed to consolidate the Young Bay Mud material. The thickness and the width of the surcharge are estimated to be 18 feet and 150 feet, respectively, to fully consolidate the Young Bay Mud layer.

The fill surcharge would be installed as shown in Figures 4-10 through 4-14.

4.1.3 Alternative 3 - Sheet Piles with Anchors

This alternative includes the installation of sheet piles with anchors. The combination of sheet piles and anchors forms a physical buttress to limit lateral displacements and prevent waste release into San Francisco Bay.

Sheet Piles

Recent technology has resulted in a watertight sheeting called Waterloo sheet piles. These sheet piles can be installed using the same equipment and techniques as conventional sheet piles, except that a watertight joint with a low permeable grout is used to interlock the sheet piles together. Vibro equipment is suitable for most soil conditions, although better results may be achieved with impact equipment in certain cohesive soils. After the cavities have been flushed, the joints are sealed with a low permeable grout.

Waterloo sheet piles, formed of sealed steel sheet piling developed in 1989 by researchers at the Waterloo Center for Groundwater Research, University of Waterloo, are configured into a groundwater containment wall. The barrier incorporates a sealed cavity at the interlocking joint between sheet piles that can be flushed clean, inspected, and then sealed after the sheet piles have been driven into the ground. The system allows for documented quality assurance and a high degree of quality control. Bulk wall hydraulic conductivity of 10^{-8} to 10^{-10} centimeters per second have typically been achieved in university-conducted testing. Waterloo sheet piles can also be used to prevent off-site migration of contaminated groundwater or gases.

This alternative would require a field investigation along the alignment of the Waterloo sheet piles to design the sheet pile. The field investigation shall consist of drilling hollow-stem auger soil borings spaced every 100 feet to a depth of 60 feet below ground surface (bgs). Standard penetration number and type of material shall be recorded at 5-foot intervals. Compatibility testing needs to be conducted to assess the compatibility of the Waterloo sheet pile material and the grout with the bay water and the soil. Soil samples may be collected during the drilling activities along with the bay water samples. The compatibility testing will include analytical and geotechnical testing on the water and the soil samples, respectively.

Waterloo sheet piles would be driven from the ground surface to approximately 60 feet. The limits and alignment of the Waterloo sheet piles are shown in Figures 4-15 and 4-16, and the

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	03136848.DWG

I:\1990-RAC\CTO-0054\DWG\031368\03136848.DWG
PLOT/UPDATE: SEP 15 2003 11:18:26

REFERENCE:
HJW-GeoSpatial, Inc., Upland Topography
NAD27, NGVD29 - CCS Zone III

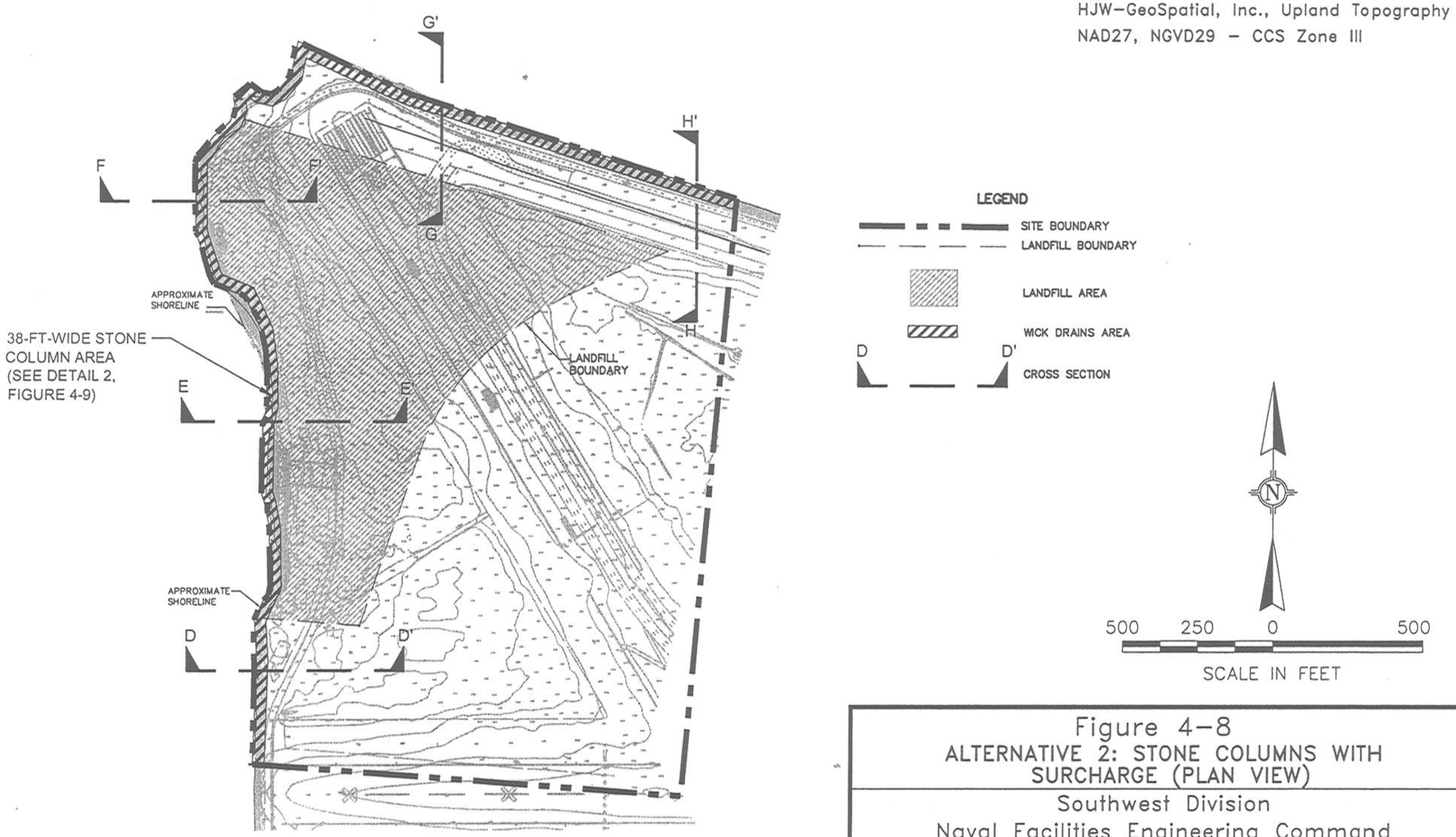
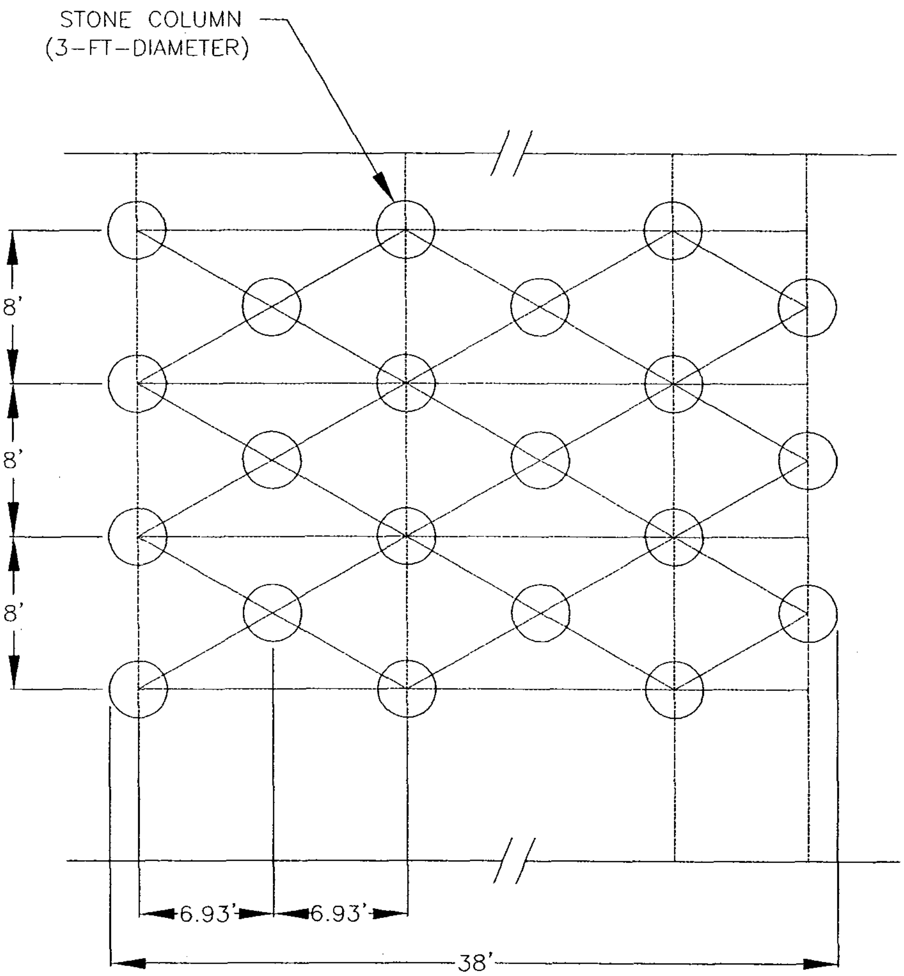


Figure 4-8
ALTERNATIVE 2: STONE COLUMNS WITH SURCHARGE (PLAN VIEW)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DETAIL 2
STONE COLUMNS CONFIGURATION



10 5 0 10
SCALE IN FEET

Figure 4-9
ALTERNATIVE 2 - STONE COLUMNS WITH SURCHARGE (DETAIL 2)

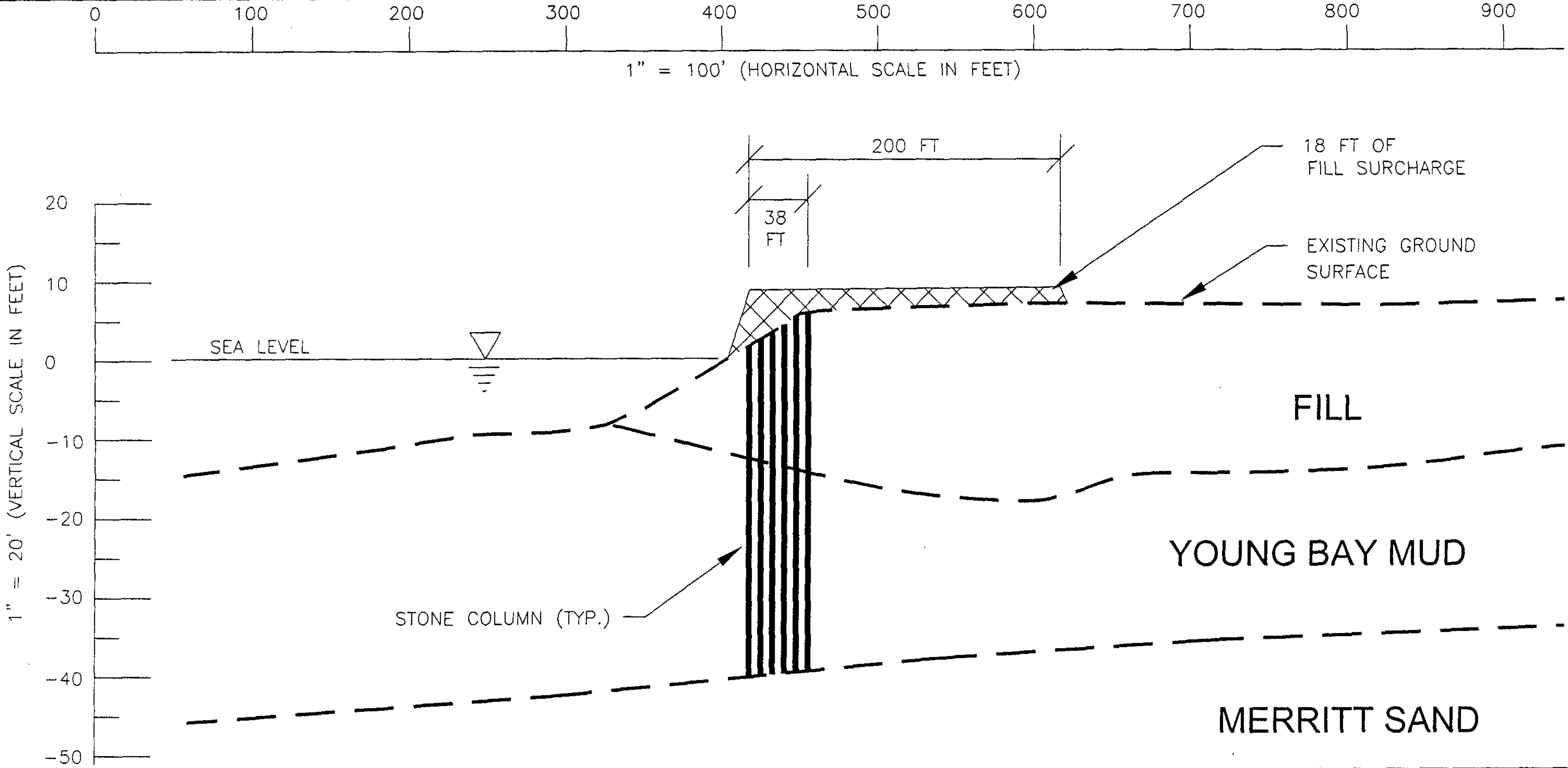
Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWN BY: M.D.	DATE: 09/26/03	
CHECKED BY: T.L.	REV: REVISION 0	
APPROVED BY: A.L.		
DCN: FWSD-RAC-03-1368	CTO: # 0054	
DRAWING NO:	03136849.DWG	

I:\1990-RAC\Cto-0054\Dwg\031368\03136849.dwg 09/15/2003 11:19:27 AM PDT

DRAWN BY: M.D.	CHECKED BY: TL	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 031368410.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION D-D' (LOOKING NORTH)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

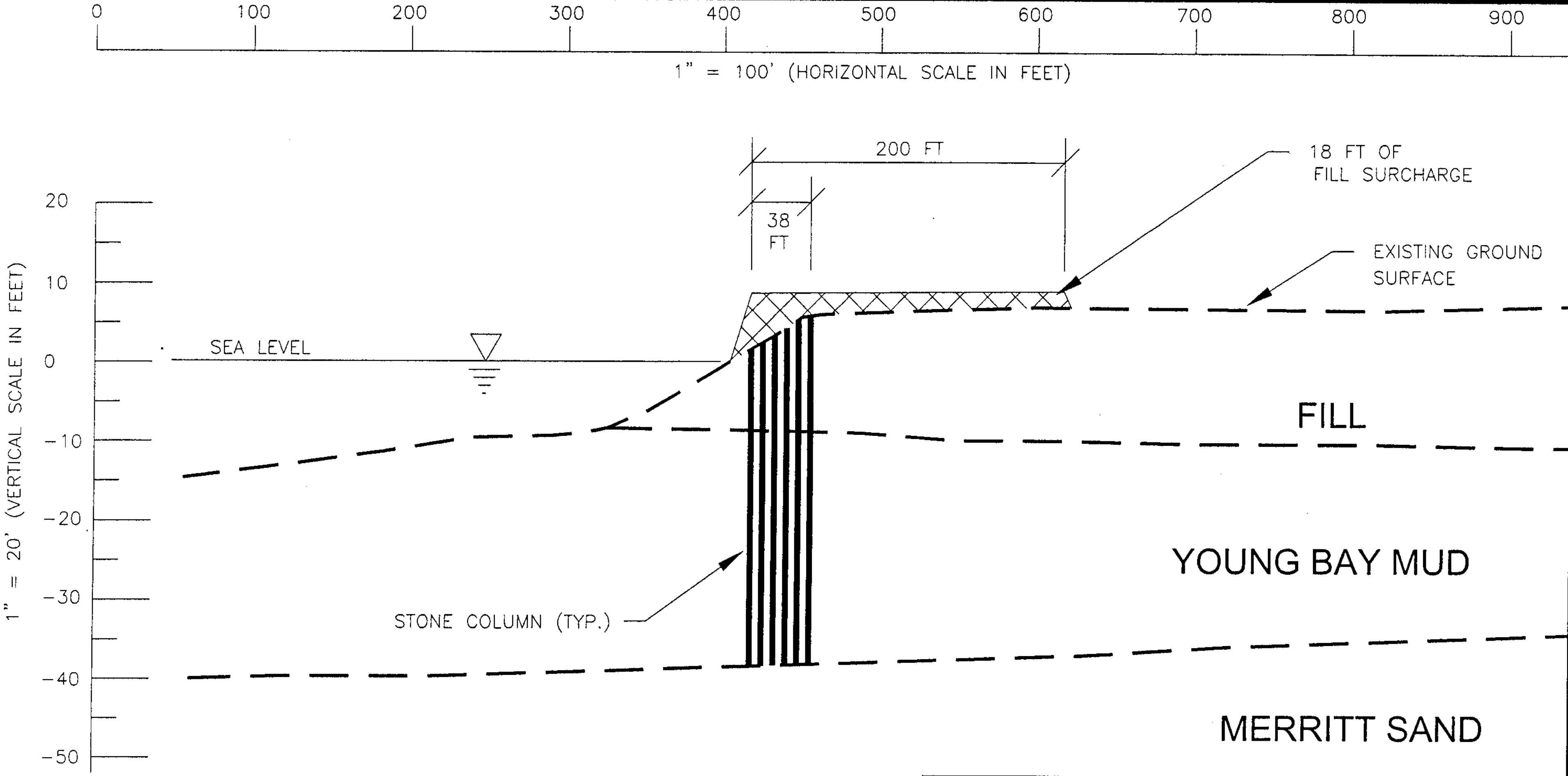
Figure 4-10
ALTERNATIVE 2 - STONE COLUMNS WITH SURCHARGE (SECTION D-D')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-10.dwg 09/12/2003 03:49:56 PM PDT

DRAWN BY: M.D.	CHECKED BY: TL	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	031368411.DWG



SECTION E-E' (LOOKING NORTH)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

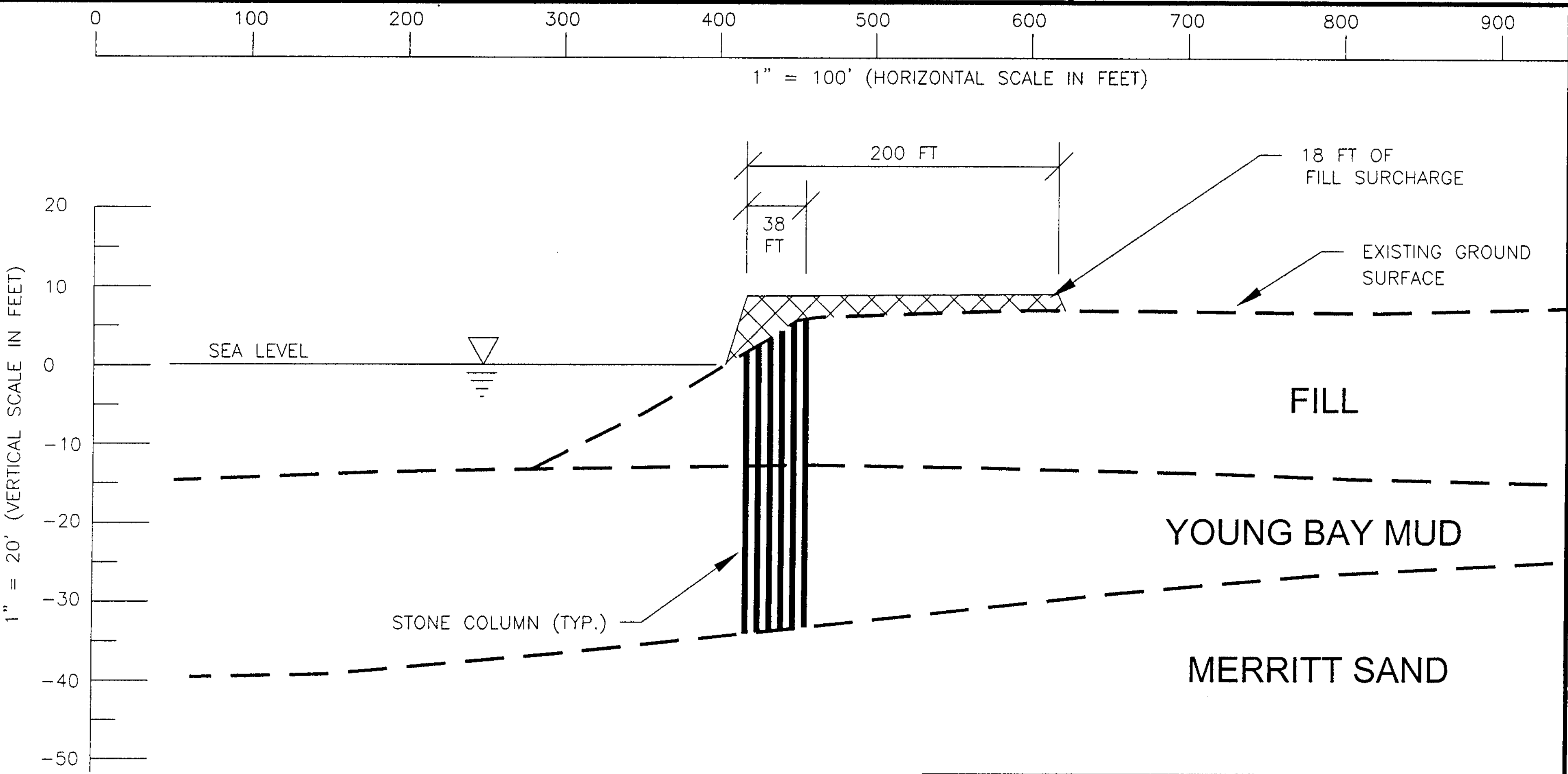
Figure 4-11
ALTERNATIVE 2 – STONE COLUMNS WITH SURCHARGE (SECTION E-E')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-11.dwg 09/12/2003 03:51:03 PM PDT

DRAWN BY: M.D.	CHECKED BY: D.M.	APPROVED BY: TL	DCN: FWSD-RAC-03-1348	DRAWING NO: 031368412.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION F-F' (LOOKING NORTH)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

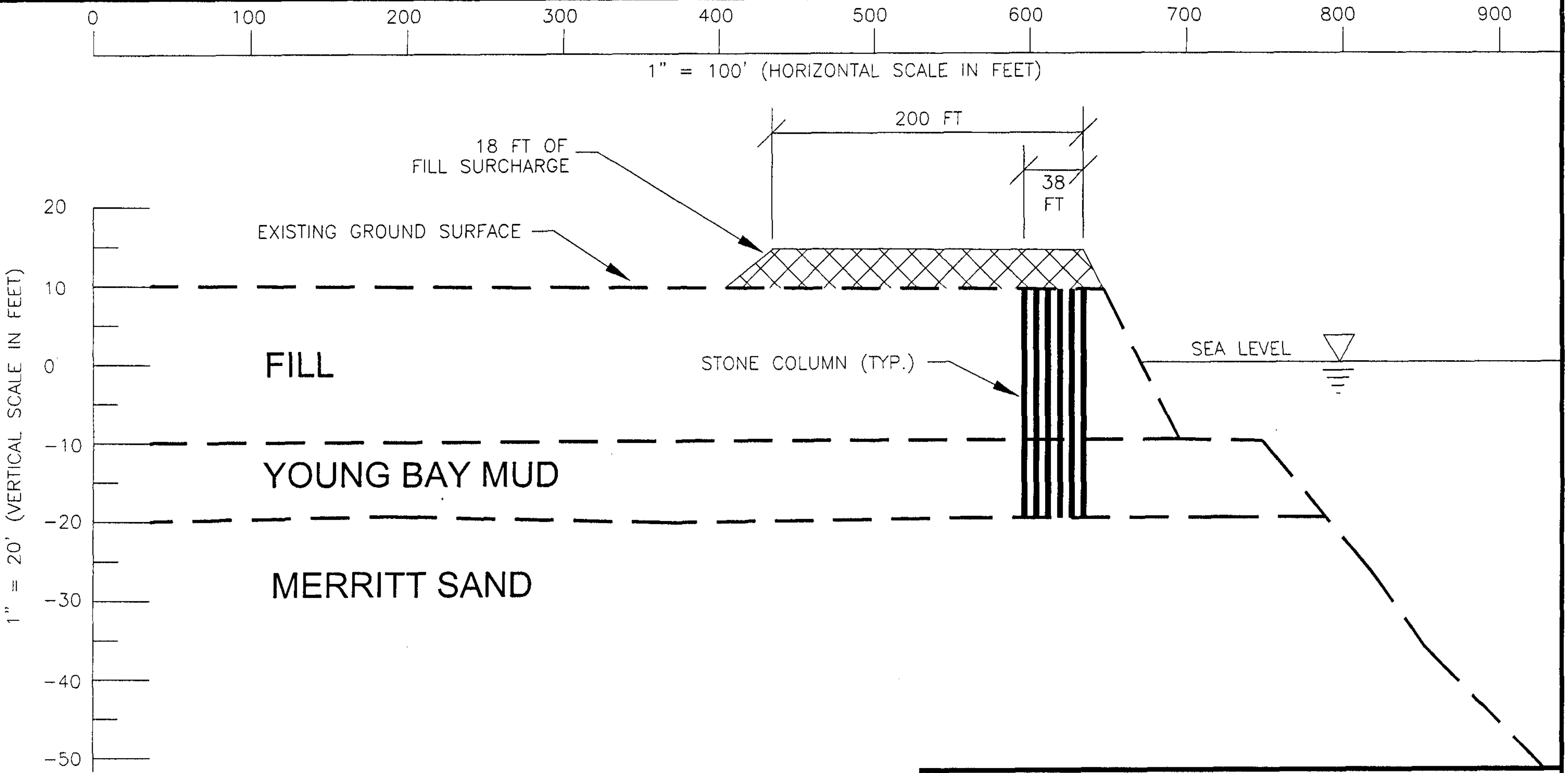
Figure 4-12
ALTERNATIVE 2 – STONE COLUMNS WITH SURCHARGE (SECTION F-F')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-12.dwg 09/12/2003 03:52:50 PM PDT

DRAWN BY: M.D.	CHECKED BY: TL	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 03136813.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: #0054	



SECTION G-G' (LOOKING WEST)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

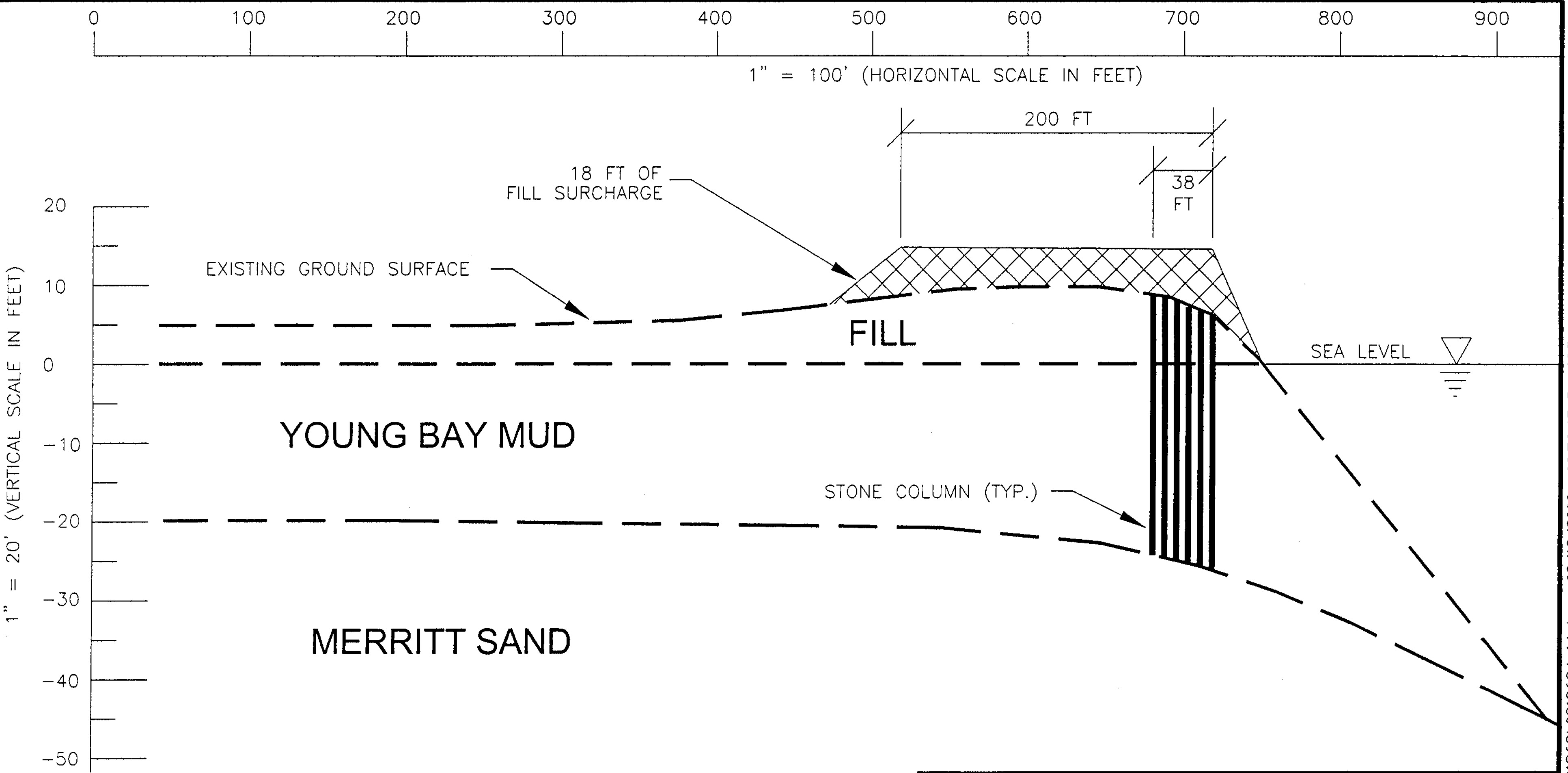
Figure 4-13
ALTERNATIVE 2 -STONE COLUMNS WITH SURCHARGE (SECTION G-G')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-13.dwg 09/12/2003 03:55:41 PM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 03136814.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION H–H' (LOOKING WEST)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

Figure 4–14
ALTERNATIVE 2 – STONE COLUMNS WITH SURCHARGE (SECTION H–H')

Southwest Division
Naval Facilities Engineering Command

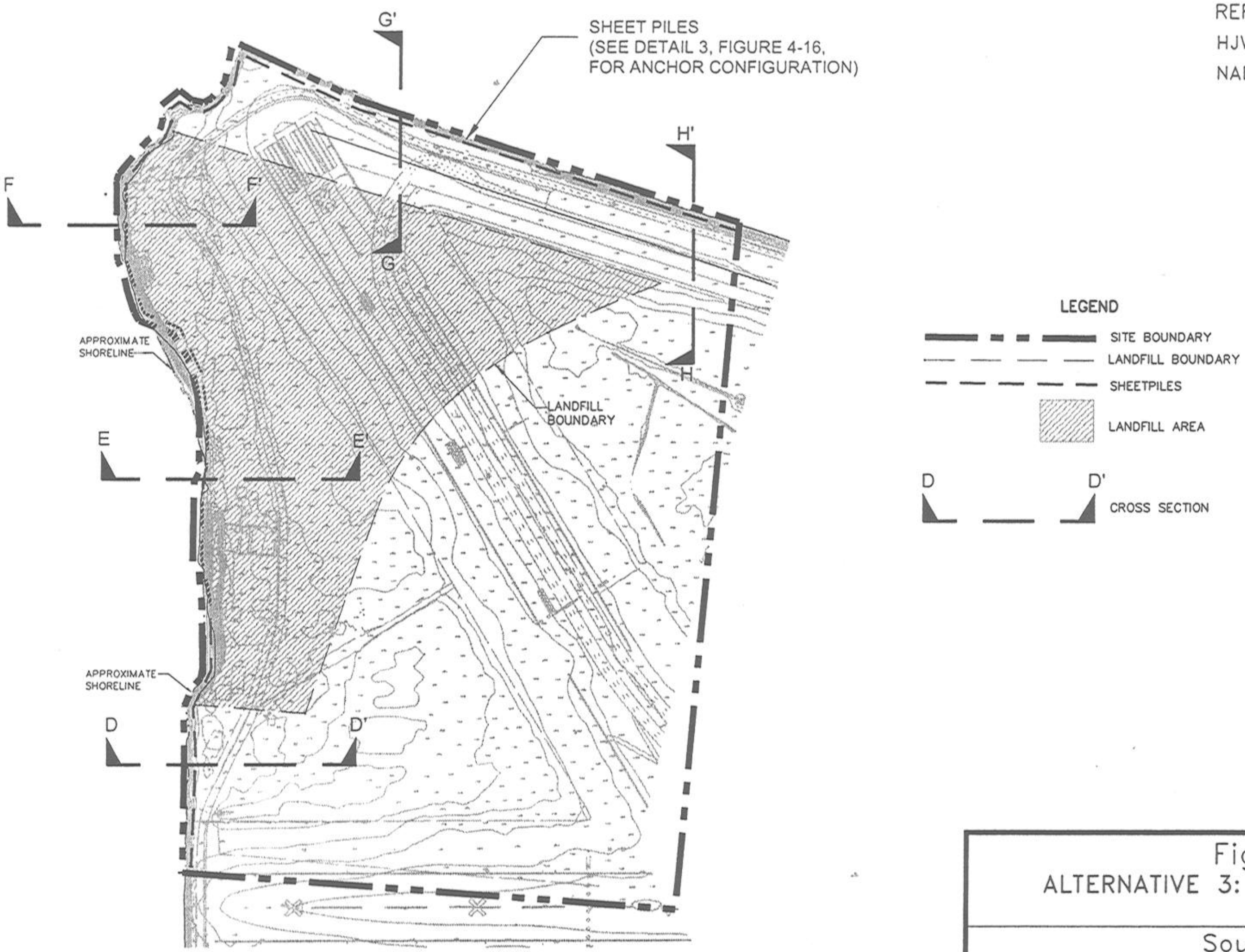
FOSTER WHEELER ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-14.dwg 09/12/2003 03:56:47 PM PDT

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRA ING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-15.DWG

I:\1990-RAC\CTO-0054\DWG\031368\0313684-15.DWG
PLOT/UPDATE: SEP 12 2003 16:09:02

REFERENCE:
HJW-GeoSpatial, Inc. Upland Topography
NAD27, NGVD29 - C :S Zone III



500 250 0 500
SCA E IN FEET

Figure 4-15
ALTERNATIVE 3: SHEET PILES WITH ANCHORS
(PLAN VIEW)

Southwest Divisic n
Naval Facilities Engineerin Command

FOSTER WHEEL ER
ENVIRONMENTAL CORP RATION

DRAWN BY: M.D. | CHECKED BY: T.L. | APPROVED BY: A.L. | DCN: FWSD-RAC-03-1368 | DRAWING NO: 031368416.DWG
DATE: 09/26/03 | REV: REVISION 0 | CTO: # 0054

DETAIL 3

ANCHOR CONFIGURATION

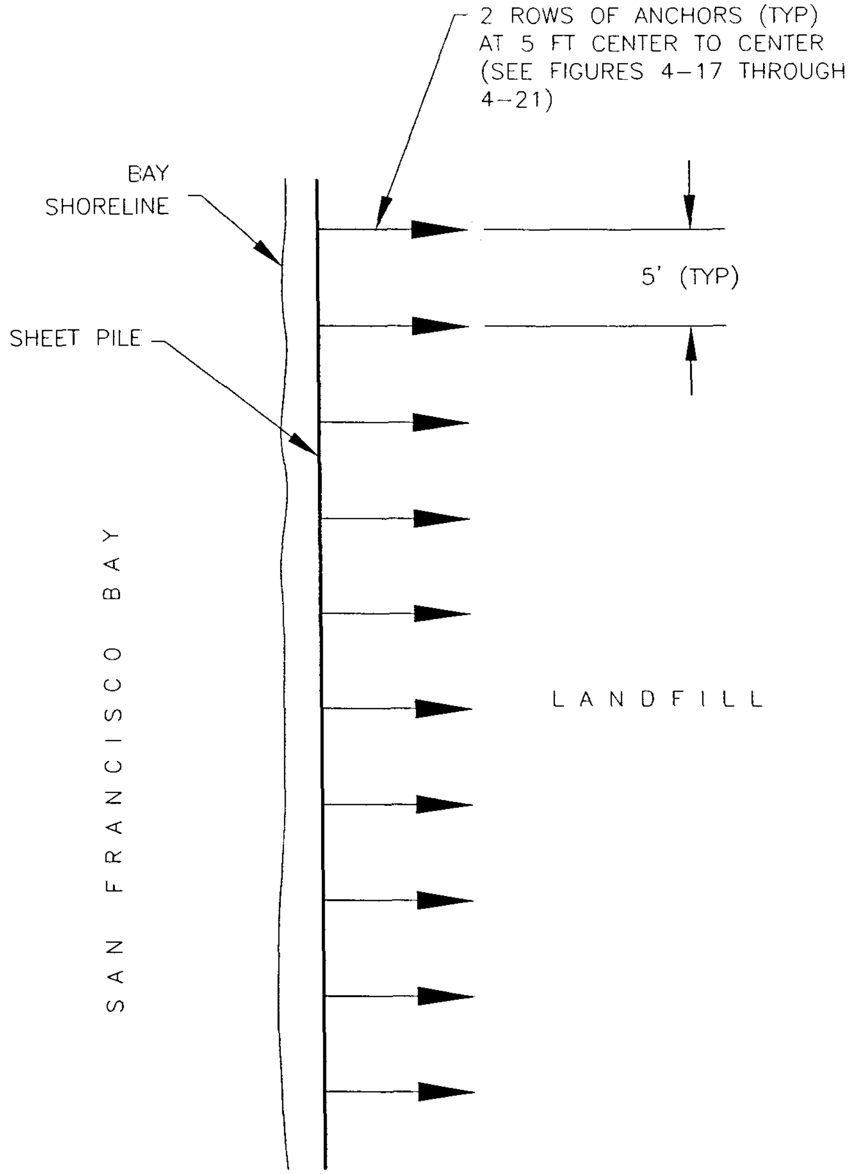


Figure 4-16

ALTERNATIVE 3-SHEET PILES WITH ANCHORS (DETAIL 3)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

cross sections are shown in Figures 4-17 through 4-21. Waterloo sheet piles would be driven using a vibratory hammer. The hollow-stem auger will not be used to install the sheet piles. Anchors would be needed to decrease deformation and stresses in sheet piles. The anchors would be spaced every 5 feet to minimize the lateral deflections.

The installation of sheet piles in soft soil layers such as the Young Bay Mud layer involves constructibility concerns regarding the vertical alignment of the sheet piles. Stringent construction quality control measures are required to ensure that the sheet piles are properly installed. In addition, sheet piles should not be used in rocky soil areas.

Anchors

Anchors are installed to support the sheet piles. Two rows of steel anchors would be placed to create a passive force as shown in Figures 4-17 through 4-21. The anchors would be spaced every 5 feet to minimize the lateral deflections.

4.1.4 Alternative 4 - Stone Columns with Surcharge and Sheet Piles

This alternative includes the installation of stone columns to accelerate the consolidation of the Young Bay Mud layer with a surcharge to consolidate the Young Bay Mud layer and Waterloo sheet piles as a containment system. The use of sheet piles in this alternative would be necessary if Alternative 2 generates greater than allowable lateral displacements, or if the width of the stone columns is narrower than in Alternative 2. There would be no advantage using this alternative if the stone columns with surcharge alternative, or the sheet pile alternative is technically feasible.

Stone Columns

The stone column description is included in Section 4.1.2. Figure 4-22 shows the alignment and location of the stone columns. Stone columns would be installed along the shoreline perimeter of the site extending down to the Young Bay Mud layer. The installation of stone columns would replace existing weak soils (fill material and Young Bay Mud) with higher strength stones (increased shear strength) and densify the surrounding soils. Commonly used methods to install stone columns include the wet top feed, dry bottom feed, and "Frankie" stone column methods as described in Section 3.1 (see initial description of Alternative 2: Stone Columns with Surcharge). The size of the stones will be determined during the design phase. The configuration of the stone columns is shown in Figure 4-23, and the cross sections are shown in Figures 4-24 through 4-28. The stone columns would extend from the shoreline to 20 feet into the upland area.

Surcharge

A surcharge consisting of import fill material would be placed to consolidate the Young Bay Mud layer. The thickness and width of the surcharge are estimated to be 5 feet and 150 feet, respectively, for partial consolidation of the Young Bay Mud layer.

Sheet Piles

The sheet piles description is provided in Section 4.1.3. The installation of sheet piles in San Francisco Bay may generate a constructibility concern regarding the vertical alignment of the sheet piles. The installation would require stringent construction quality control measures to ensure that the sheet piles are installed properly.

4.1.5 Alternative 5 - Soil Cement Gravity Wall and Stone Columns

This alternative includes the installation of a soil cement gravity wall to increase the shear strength of the Young Bay Mud layer and stone columns to densify the fill layer.

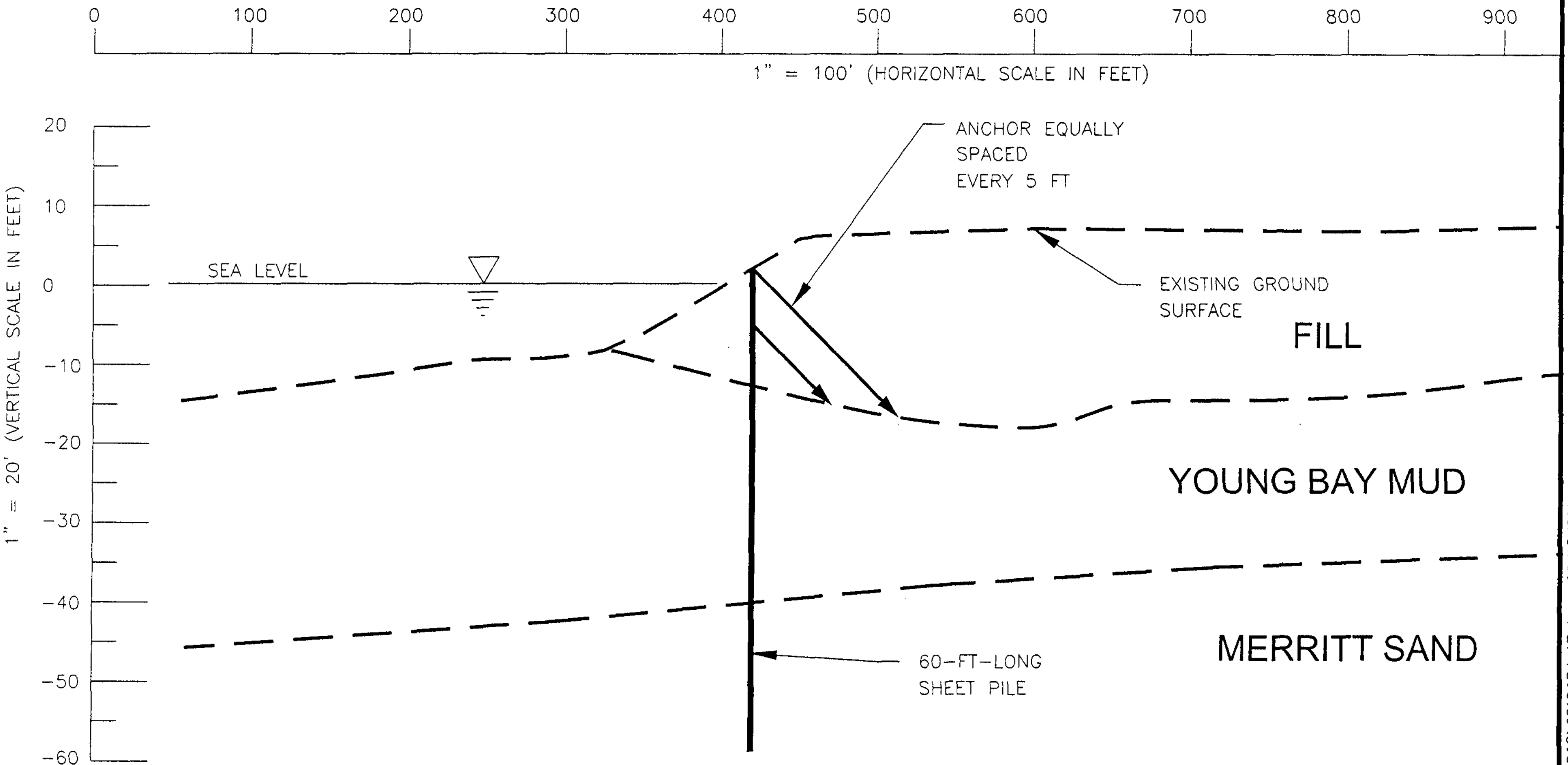
Soil Cement Gravity Wall

The soil cement gravity wall is constructed using a deep soil mixing technique, which changes the physical characteristics of the Young Bay Mud soils. Soils are converted in place to a stable mixture. Large-diameter augers are used to inject stabilizing agents, such as cement slurry, and to mix with the Young Bay Mud material. The presence of rocks and obstructions in the fill or Young Bay Mud layer will limit the effectiveness of this alternative.

The soil mixing system makes use of a crane-supported set of leads that guide a series of hydraulically driven mixing paddles and augers. As the ground is penetrated, a cement grout, stored via pigs or silos and mixed at the batch plant, will be fed through the center of each shaft. The auger flights loosen the soil to mix and remix it with paddles, which blend the cement with the soil. As the augers advance to a greater depth, the soil and cement are remixed by the additional mixing paddles on each shaft. When the desired depth is reached, the augers would be withdrawn and the mixing process would be repeated on return to the surface. A continuous wall or stabilized block of soil would be left behind without removing material, resulting in treatment of existing Young Bay Mud soils. Due to the spacing of the shafts, there will be continuous overlap with adjacent soil columns.

Advantages of the soil mixing method over other conventional methods are: 1) the cutoff wall can be constructed in very soft soil conditions, whereas a conventional slurry cutoff wall trench might fail during construction; and 2) construction of the wall does not require any soil to be excavated and disposed of during construction.

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	031368417.DWG



SECTION D-D' (LOOKING NORTH)

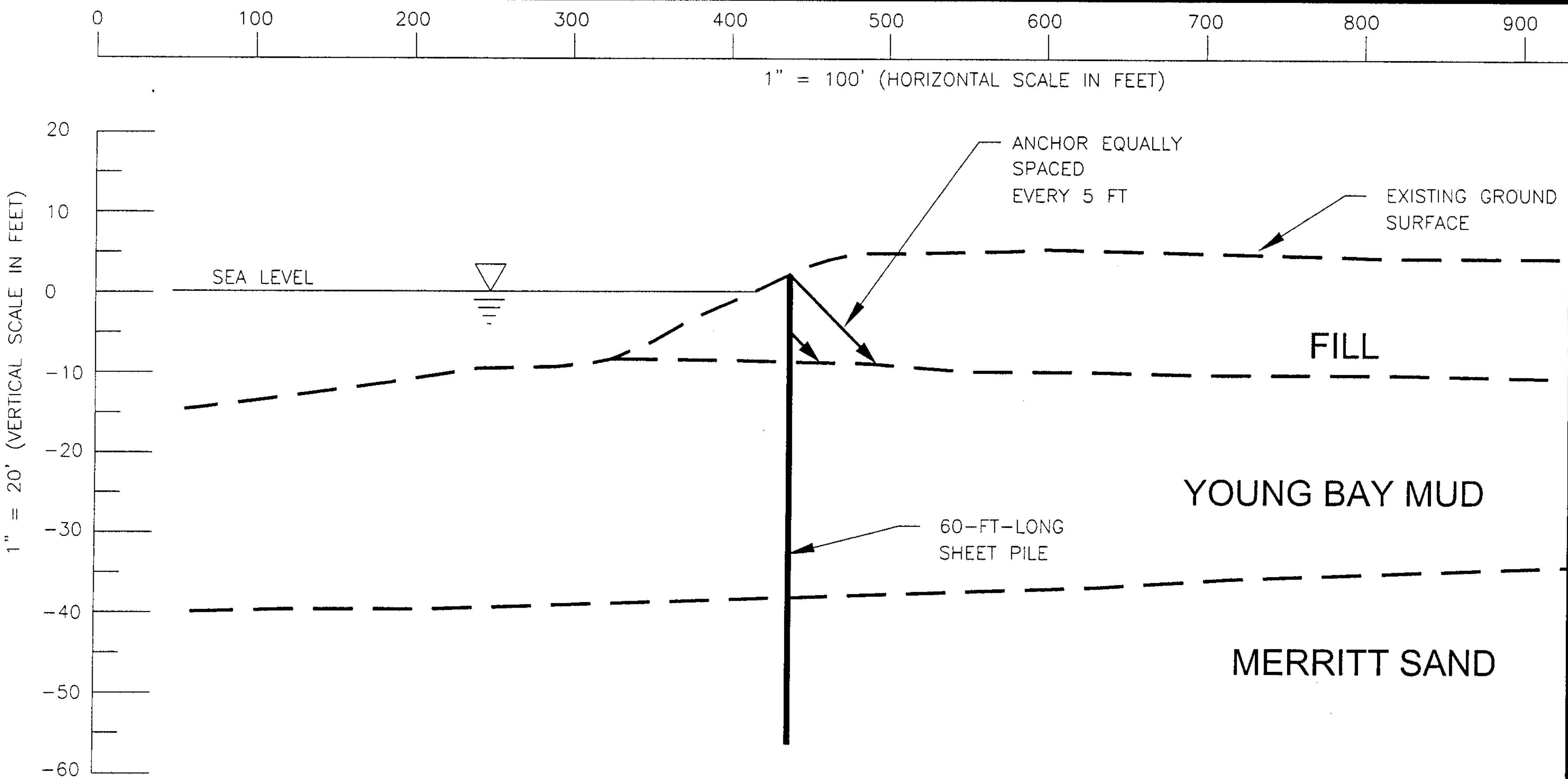
Figure 4-17
ALTERNATIVE 3 – SHEET PILES WITH ANCHORS (SECTION D-D')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-17.dwg 09/12/2003 04:17:15 PM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-18.DWG



SECTION E-E' (LOOKING NORTH)

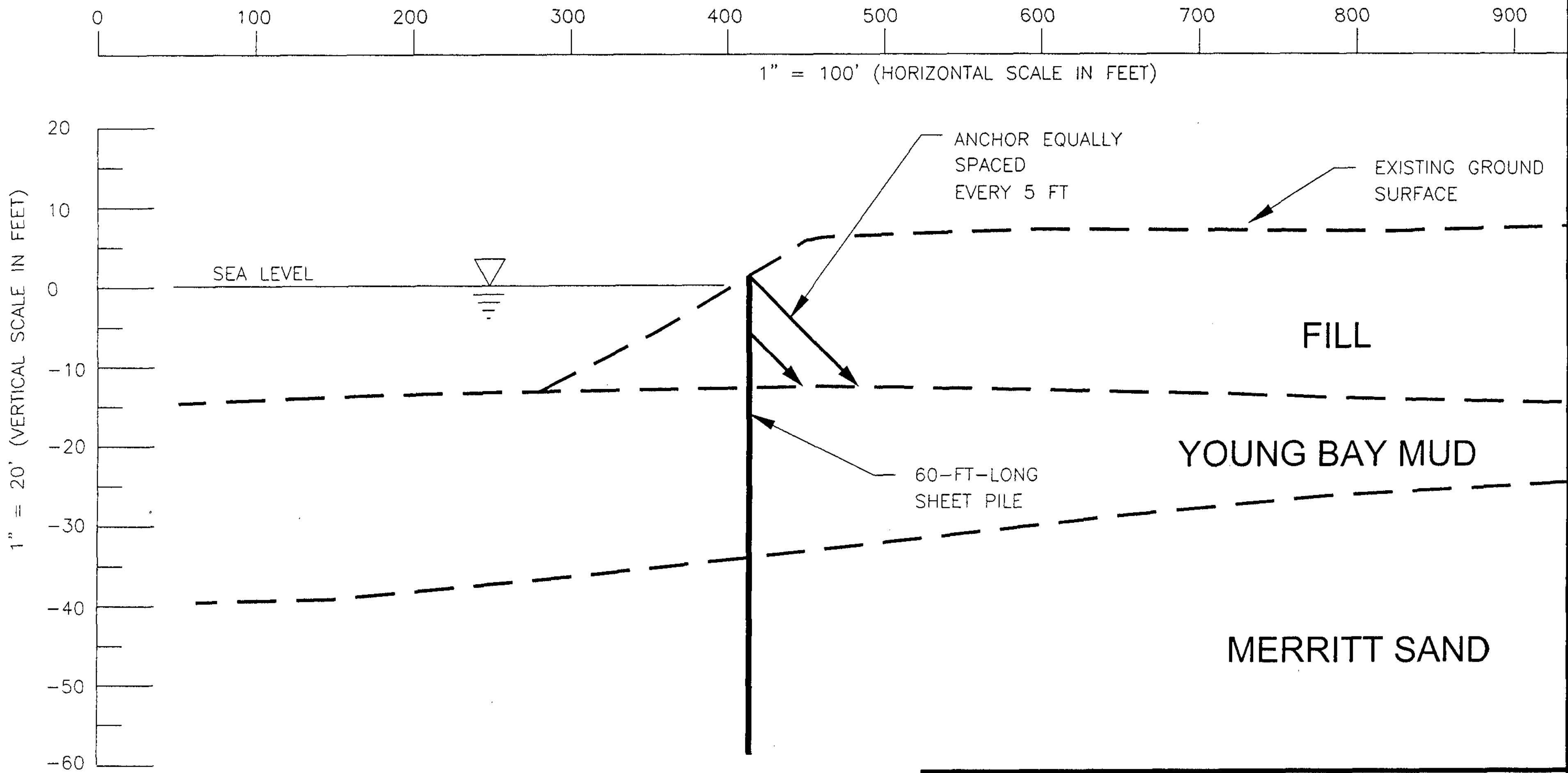
Figure 4-18
ALTERNATIVE 3 – SHEET PILES WITH ANCHORS (SECTION E-E')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-18.dwg 09/12/2003 04:18:05 PM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-19.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION F-F' (LOOKING NORTH)

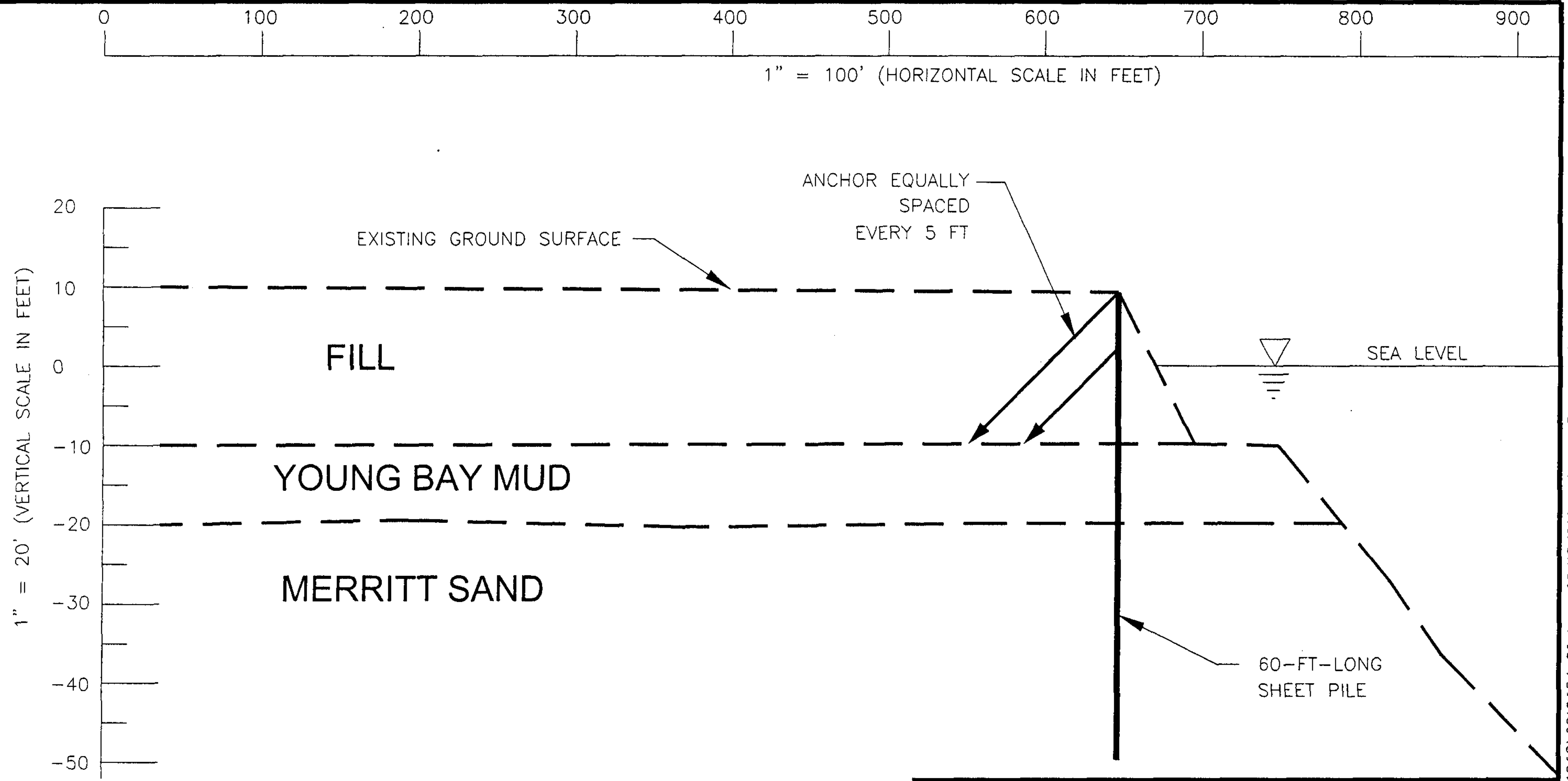
Figure 4-19
ALTERNATIVE 3 – SHEET PILES WITH ANCHORS (SECTION F-F')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-19.dwg 09/12/2003 04:20:46 PM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 031368420.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



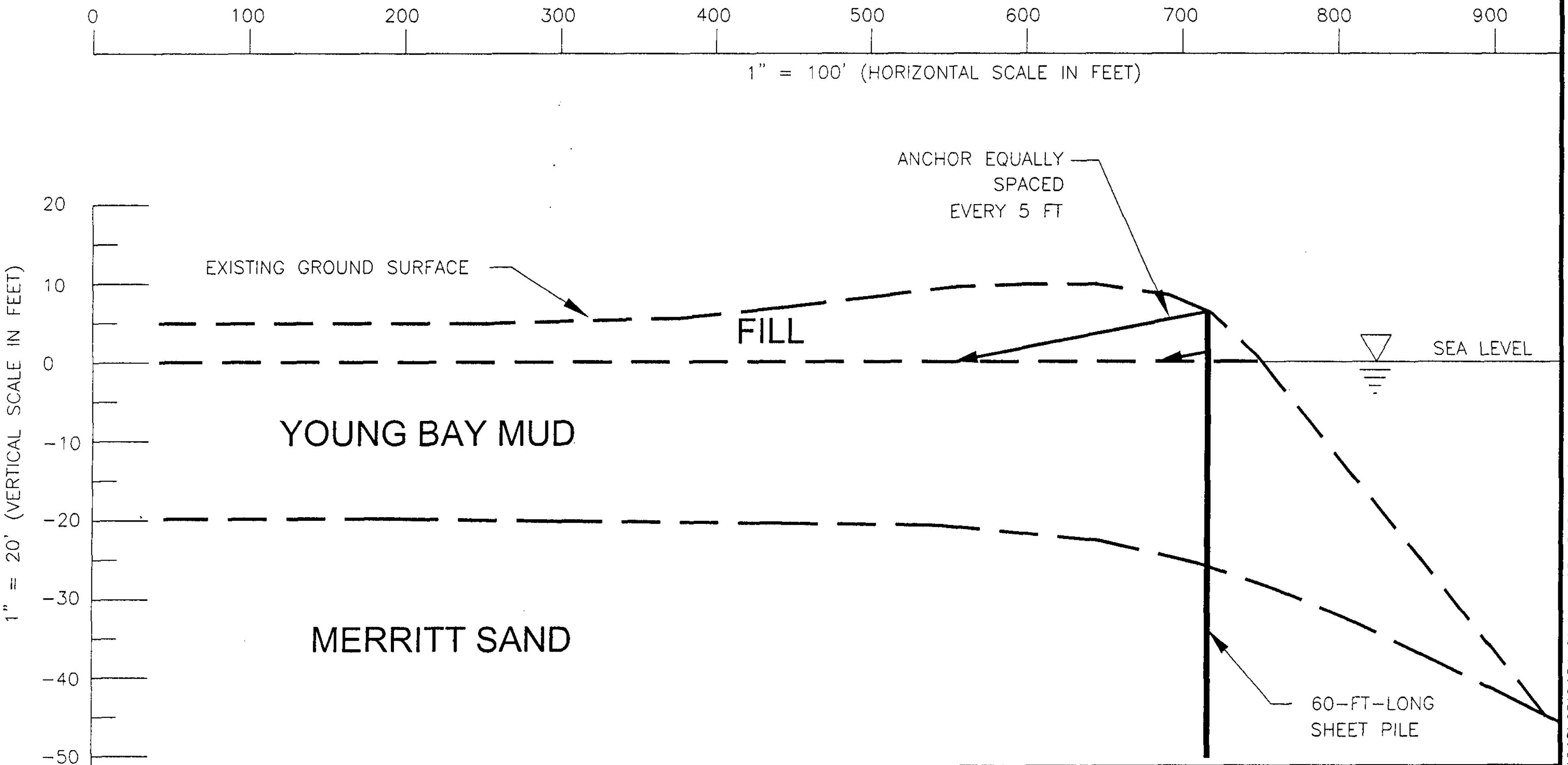
SECTION G-G' (LOOKING WEST)

Figure 4-20
ALTERNATIVE 3 – SHEET PILES WITH ANCHORS (SECTION G-G')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 03136821.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION H-H' (LOOKING WEST)

Figure 4-21
ALTERNATIVE 3 – SHEET PILES WITH ANCHORS
(SECTION H-H')

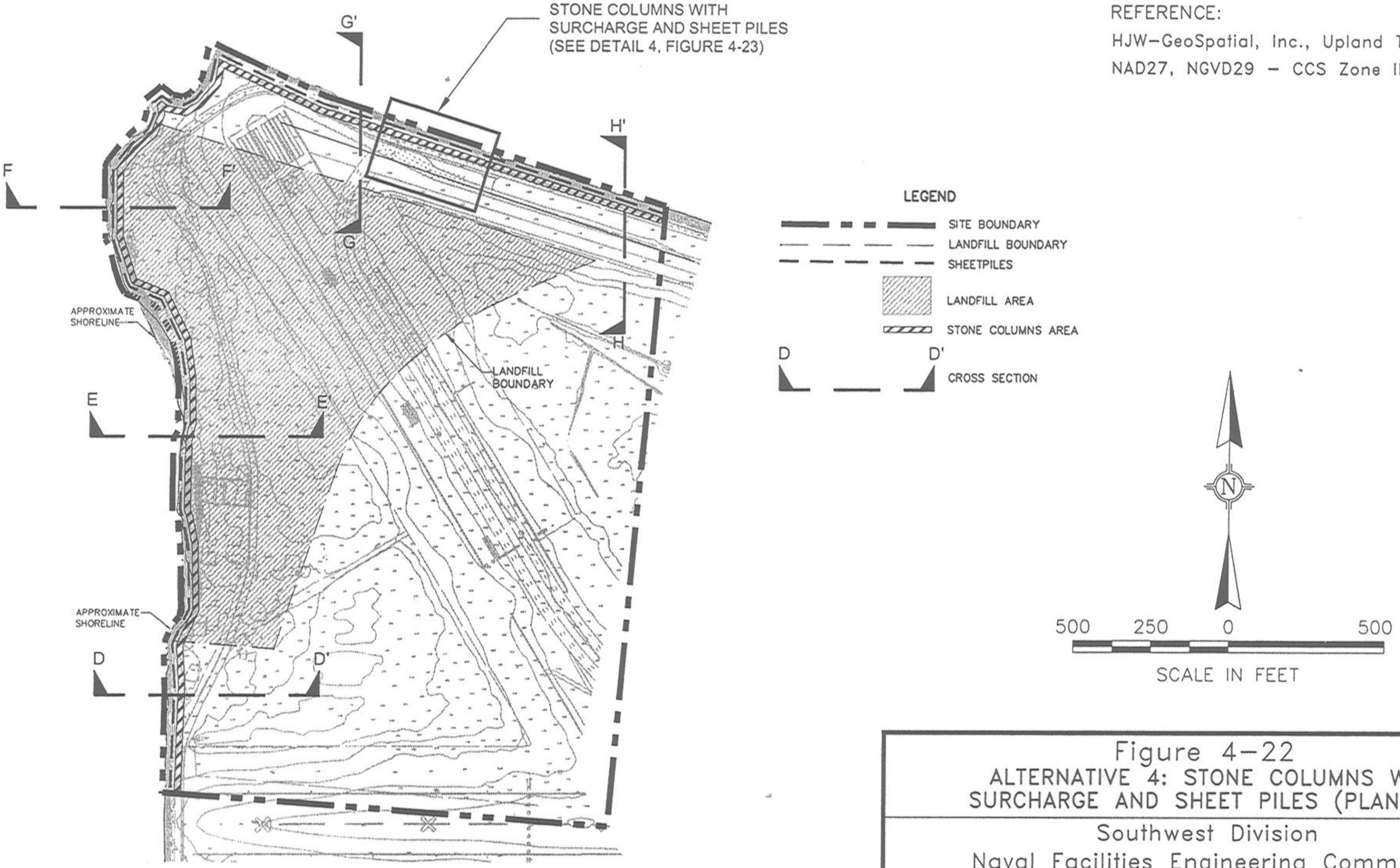
Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313368\0313684-21.dwg 09/12/2003 04:24:02 PM PDT

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-22.DWG

I:\1990-RAC\CTO-0054\DWG\031368\0313684-22.DWG
PLOT/UPDATE: SEP 12 2003 16:25:01



REFERENCE:
HJW-GeoSpatial, Inc., Upland Topography
NAD27, NGVD29 – CCS Zone III

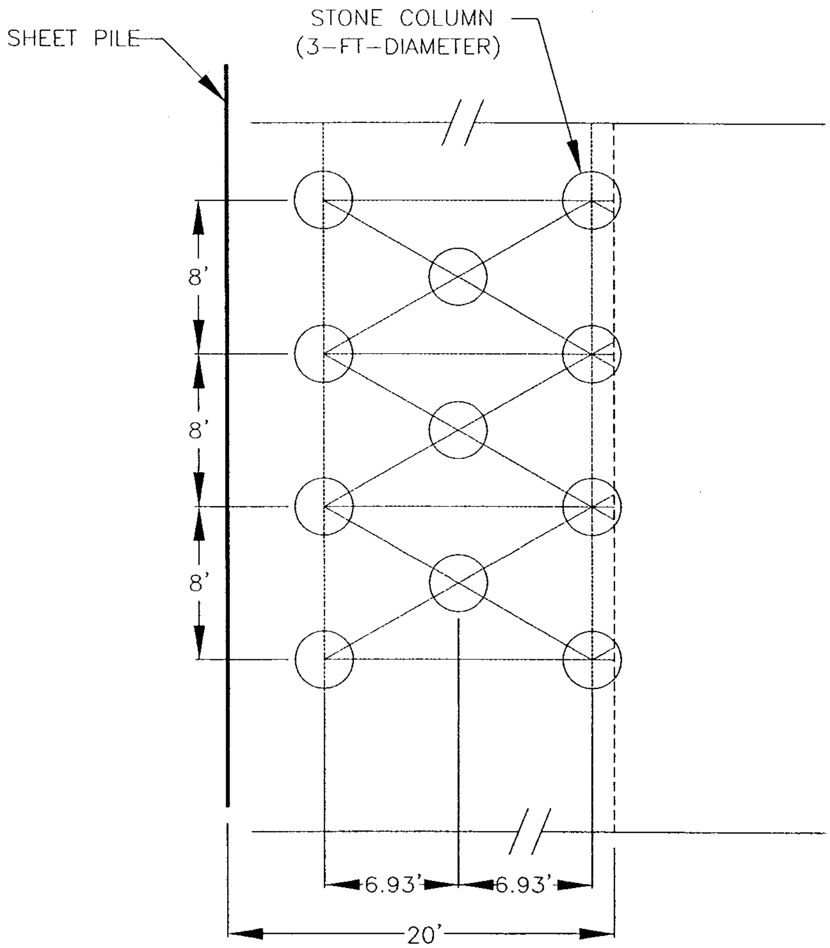
Figure 4–22
ALTERNATIVE 4: STONE COLUMNS WITH SURCHARGE AND SHEET PILES (PLAN VIEW)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DETAIL 4

STONE COLUMNS CONFIGURATION



10 5 0 10'

SCALE IN FEET

Figure 4-23

ALTERNATIVE 4 – STONE COLUMNS WITH SURCHARGE AND SHEET PILES (DETAIL 4)

Southwest Division
Naval Facilities Engineering Command

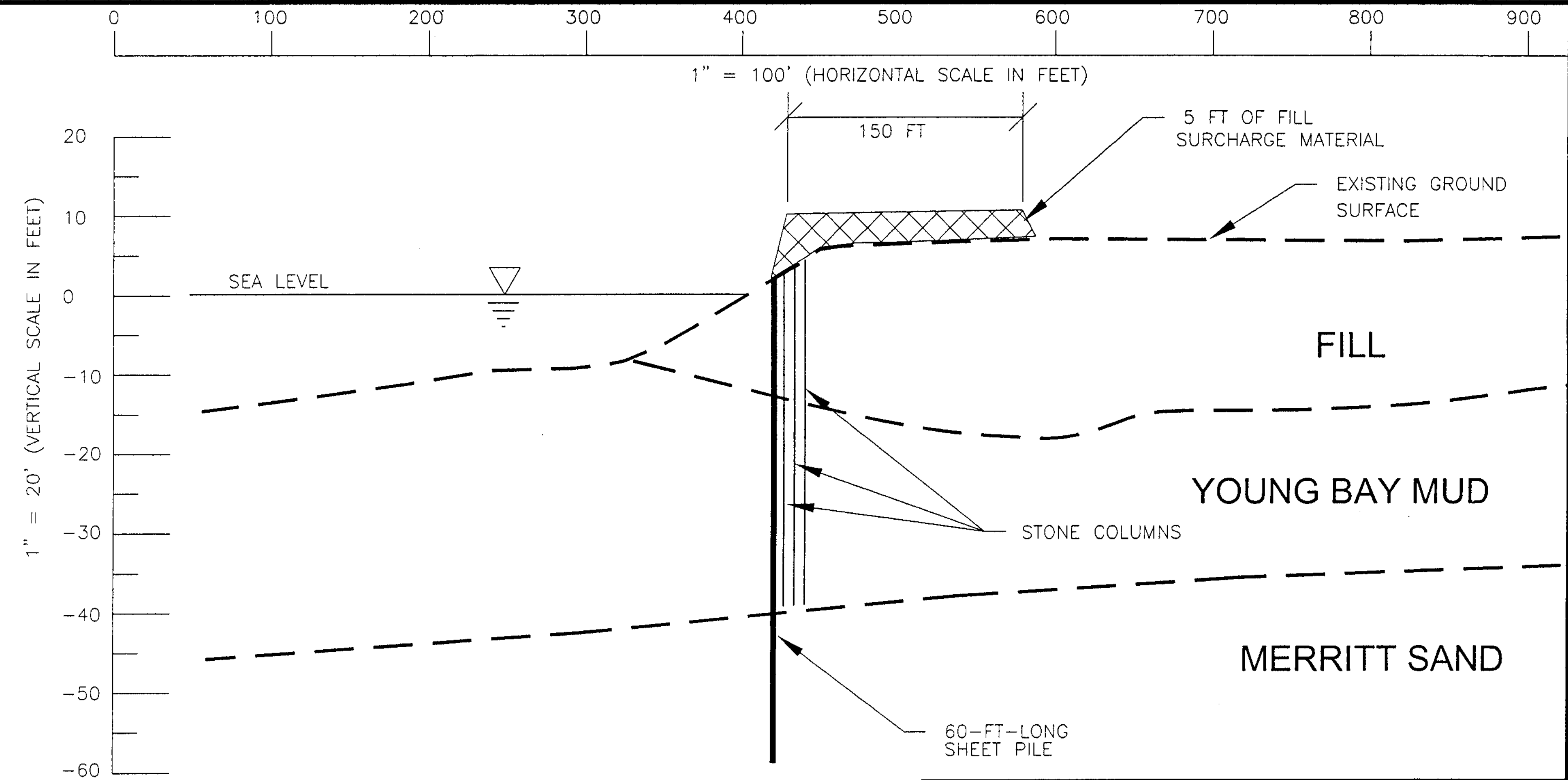
FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWN BY: M.D. | CHECKED BY: T.L. | APPROVED BY: A.L. | DCN: FWSD-RAC-03-1368 | DRAWING NO: 031368423.DWG

DATE: 09/26/03 | REV: REVISION 0 | CTO: # 0054

I:\1990-RAC\Cto-0054\Dwg\0313684-23.dwg 09/12/2003 04:26:46 PM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-24.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION D-D' (LOOKING NORTH)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

Figure 4-24

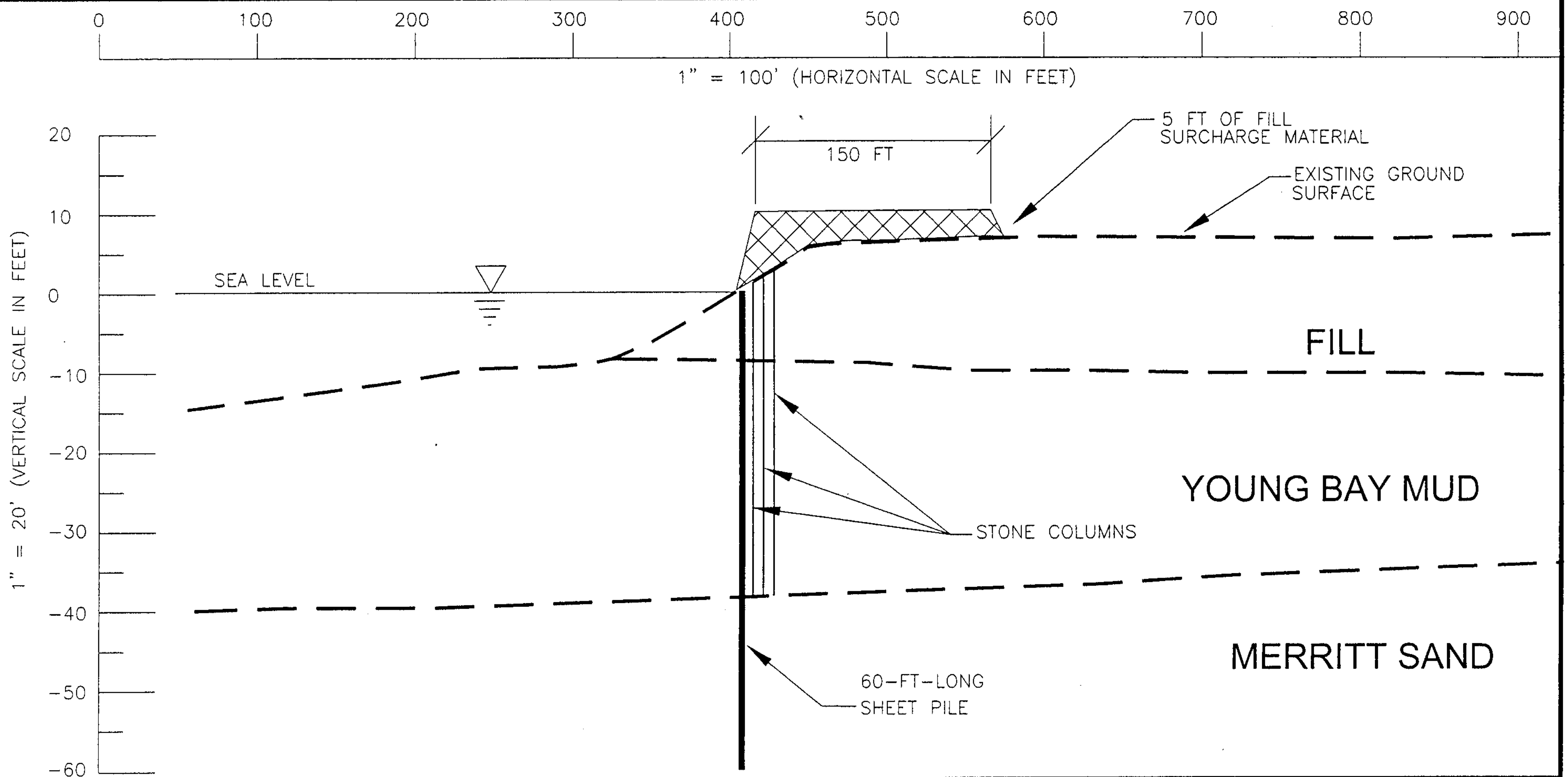
ALTERNATIVE 4 – STONE COLUMNS WITH SURCHARGE AND SHEET PILES (SECTION D-D')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-24.dwg 09/12/2003 04:29:11 PM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 031368425.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION E-E' (LOOKING NORTH)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

Figure 4-25
ALTERNATIVE 4 – STONE COLUMNS WITH SURCHARGE AND SHEET PILES (SECTION E-E')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-25.dwg 09/12/2003 04:30:09 PM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-26.DWG

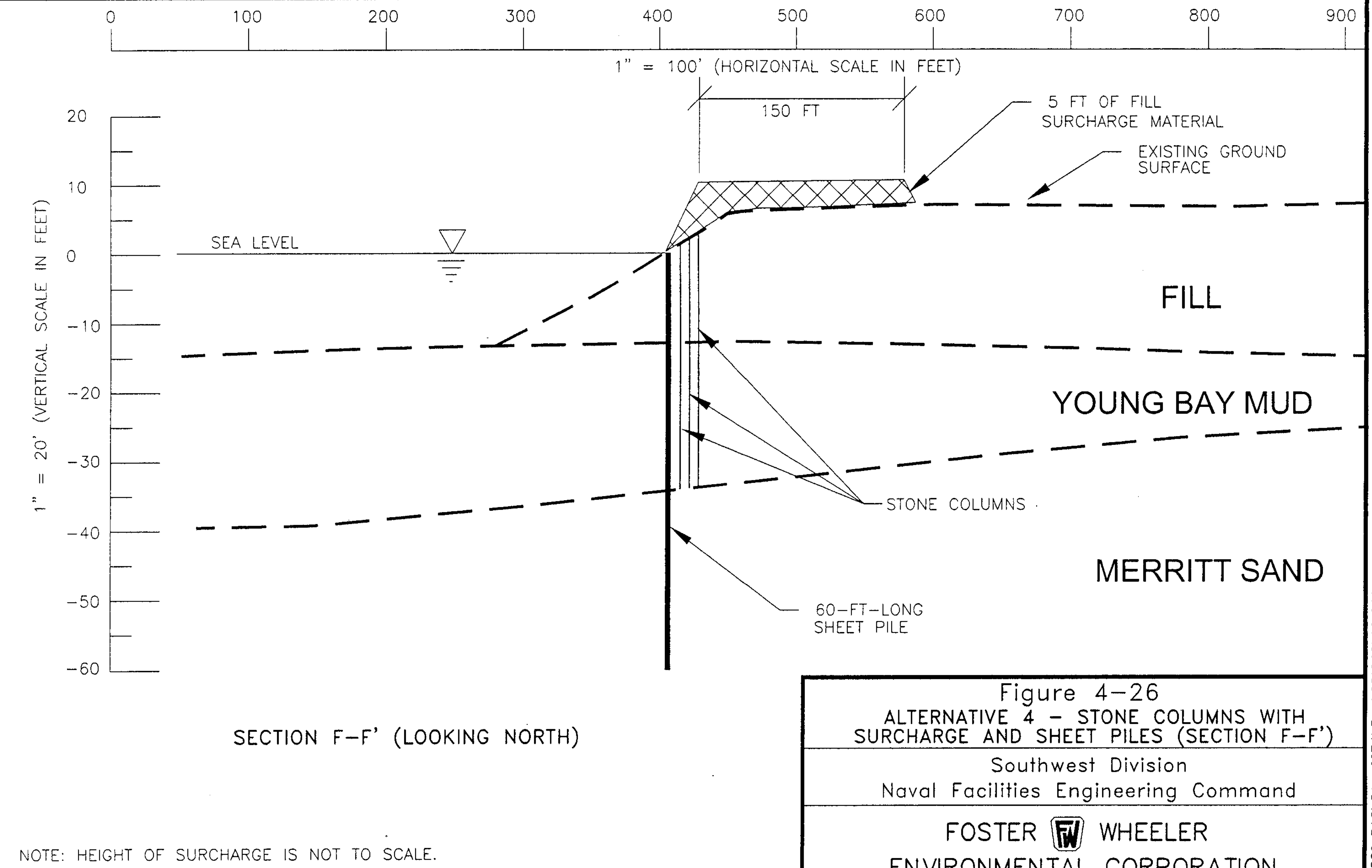


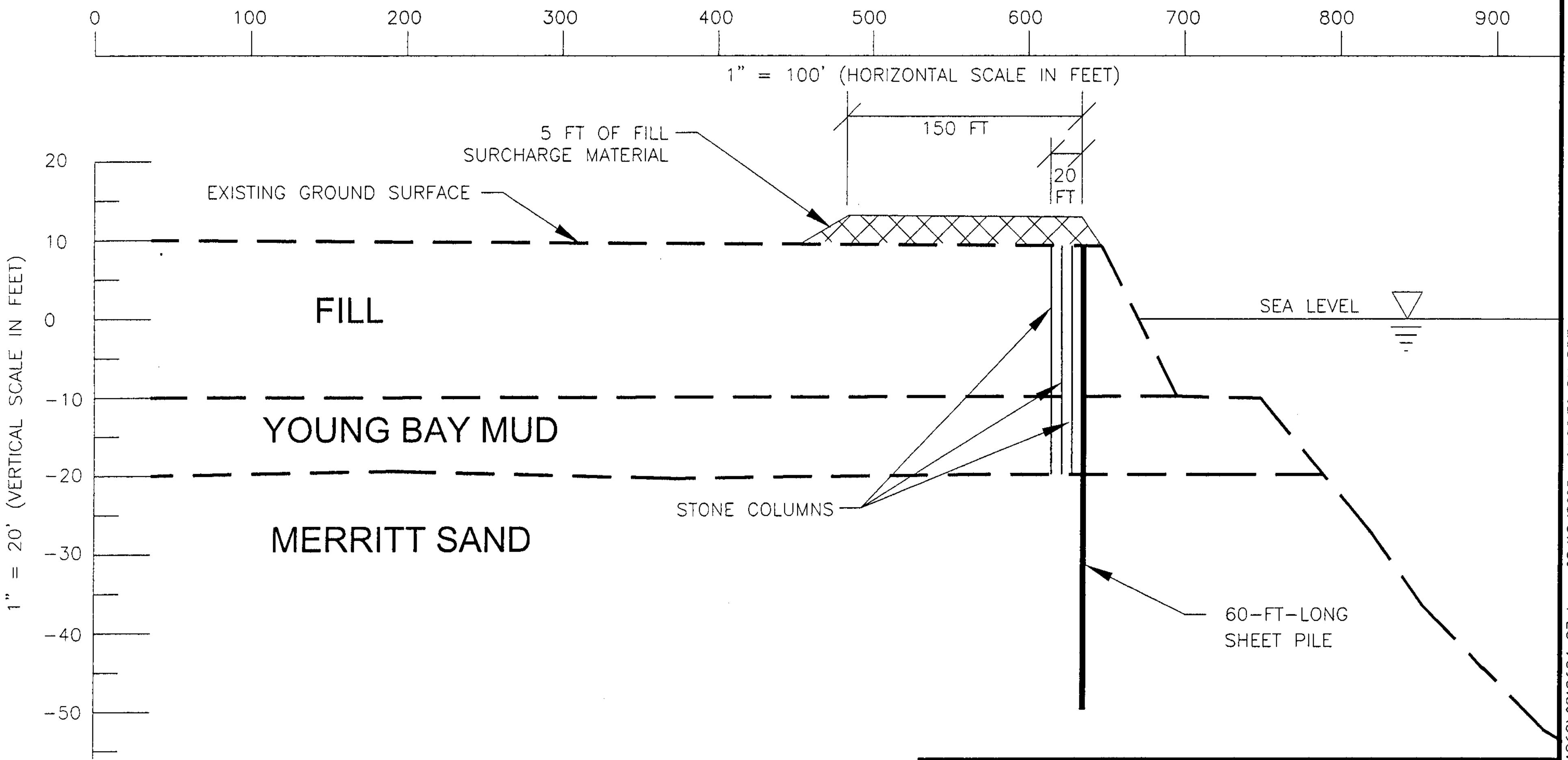
Figure 4-26
ALTERNATIVE 4 - STONE COLUMNS WITH SURCHARGE AND SHEET PILES (SECTION F-F')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-26.dwg 09/12/2003 04:31:10 PM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-27.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION G-G' (LOOKING WEST)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

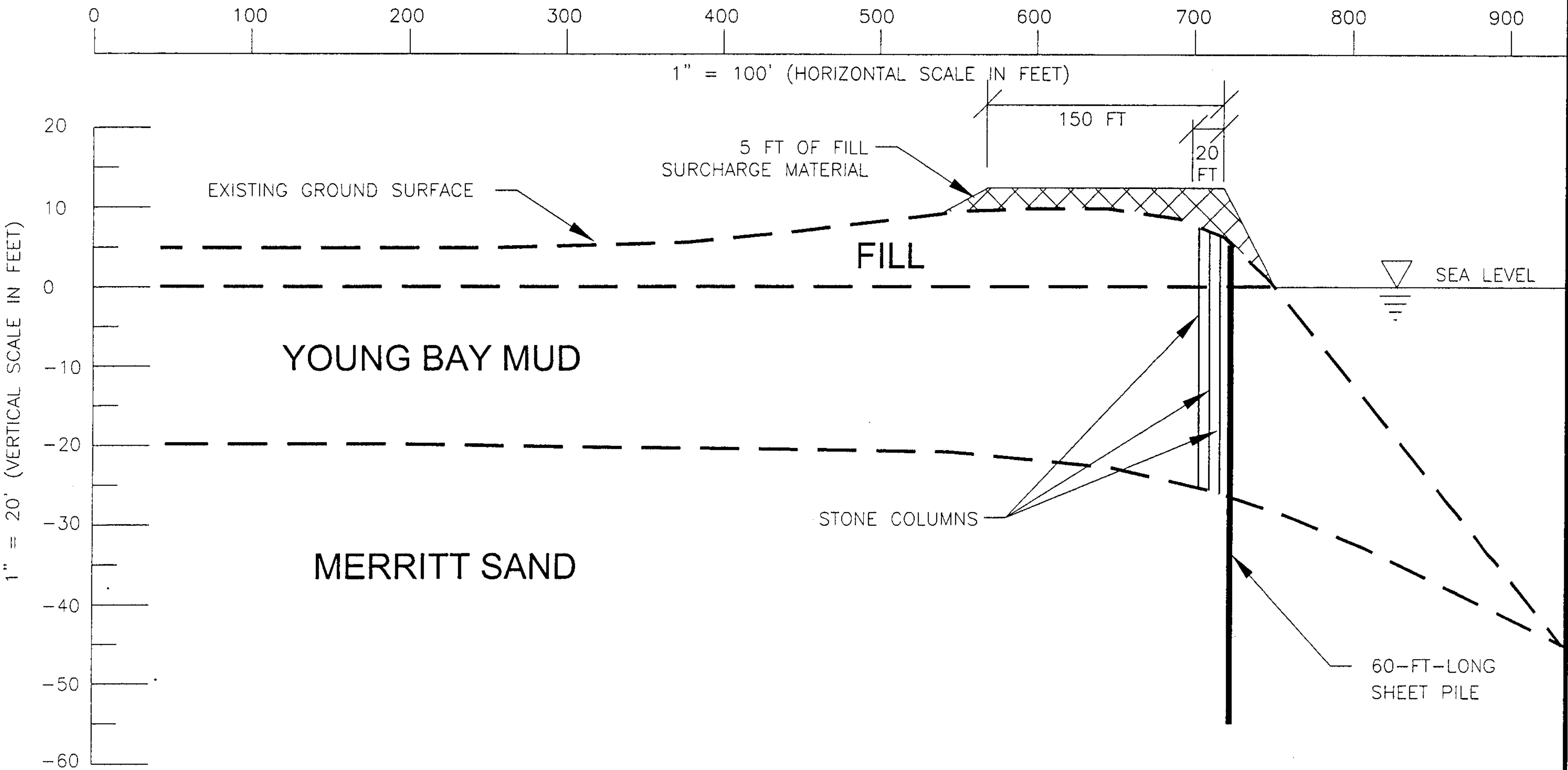
Figure 4-27
ALTERNATIVE 4 – STONE COLUMNS WITH SURCHARGE AND SHEET PILES (SECTION G-G')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-27.dwg 09/12/2003 04:33:50 PM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-28.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION H-H' (LOOKING WEST)

NOTE: HEIGHT OF SURCHARGE IS NOT TO SCALE.

Figure 4-28
ALTERNATIVE 4 – STONE COLUMNS WITH SURCHARGE AND SHEET PILES (SECTION H-H')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684\0313684-28.dwg 09/15/2003 09:58:38 AM PDT

This alternative involves both field and laboratory investigation. Samples from the Young Bay Mud layer would be collected for bench scale and compatibility testing. The bench scale testing is conducted to generate different soil cement mixes. The unconfined compressive strength for each soil cement sample is measured to determine the amount of cement needed to achieve the required compressive strength of the soil cement mix. Once the mix is selected, compatibility testing can be performed to ensure that the soil cement mix will not degrade when in contact with groundwater.

The soil cement gravity wall would be constructed from the shoreline to 24 feet upland along the shoreline perimeter as shown in Figures 4-29 and 4-30. The soil cement mixture will be performed from the top of the Young Bay Mud layer to 5 feet into the Merritt Sand layer as shown in Figures 4-31 through 4-35.

Stone Columns

The description of the stone columns is the same as described in Alternative 2, except that the stone columns would be installed only in the fill layer (Figures 4-31 through 4-35).

4.1.6 Alternative 6 - Concrete Wall

This alternative involves the installation of a concrete wall to create a physical buttress.

A trench would be excavated using a slide rail system, which supports the sidewalls during excavation. The trench is backfilled using concrete with a minimum compressive strength of 3,000 pounds per square inch (psi).

The concrete wall is constructed from the shoreline to 14 feet upland along the shoreline perimeter as shown in Figures 4-36 and 4-37. The concrete wall will be installed from the ground surface to 5 feet into the Merritt Sand layer as shown in Figures 4-38 through 4-42. Excavation and backfilling operations would be conducted in 50-foot-long sections to facilitate construction.

4.1.7 Alternative 7 - Excavation with Riprap

This alternative includes slope excavation and replacement with riprap material.

Excavation

The existing material would be excavated to the bottom of the Young Bay Mud layer along the shoreline perimeter and would extend 38 feet into the upland area as shown in Figure 4-43. The excavation would be performed from the existing ground surface to 5 feet into the Merritt Sand layer. The slope of the existing material is not known at this time. The excavated material would be placed in the existing landfill cap area and temporarily capped with 2 feet of fill material. The excavated material is assumed to be non-hazardous.

Riprap Replacement

The excavated areas would be backfilled with riprap to provide stability and to partially consolidate the Young Bay Mud layer. The depth of the riprap is shown in Figures 4-44 through 4-48.

4.1.8 Alternative 8 - Drilled Concrete Piers with Stone Columns

This alternative includes the installation of drilled concrete piers to create a physical buttress and the use of stone columns to increase the shear strength of the fill and Young Bay Mud layers.

Drilled Concrete Piers

Drilled concrete piers would be constructed by excavating 3-foot-diameter boreholes spaced 8 feet center to center. The concrete piers would be installed in two rows at the shoreline and along the alignment of the shoreline as shown in Figure 4-49. The piers would be drilled from the ground surface to approximately 60 feet deep. The cross sections at different locations are shown in Figures 4-50 through 4-54. The excavated soil would be placed and temporarily capped with 2 feet of fill material in the existing landfill area. Once the borehole is drilled or excavated and steel reinforcement is placed, it would be immediately filled with concrete.

Compatibility testing shall be performed to assess the degradation effect of the concrete with the bay water. The testing would include long-term permeability testing of the concrete sample permeated with bay water. Different types of cement (Type I, II, III, IV, V) would be used for the compatibility testing.

Stone Columns

Stone columns would be installed in the fill layer between the two rows of drilled concrete piers, as shown in Figures 4-50 through 4-54. The stone columns would be spaced on 5-foot centers and would be 3 feet in diameter. Detailed installation procedures are described in Section 4.1.2.

4.1.9 Alternative 9 - Pre-cast Concrete Piles

This alternative includes the installation of pre-cast concrete piles to create a physical buttress.

Pre-cast concrete piles would be 2 feet in diameter and spaced 6 feet center to center. Four rows of pre-cast concrete piles would be installed along the alignment of the shoreline as shown in Figure 4-55. Spacing of the rows would be 4 feet center to center. The pre-cast concrete piles would be driven through the ground using an impact hammer. The length of each pre-cast concrete pile would be 60 feet as shown in Figures 4-56 through 4-60.

Compatibility testing should be performed to assess the degradation effect of the concrete with the bay water. The testing would include long-term permeability testing of the concrete sample

I:\1990-RAC\CTO-0054\DWG\031368\0313684-29.DWG
PLOT/UPDATE: SEP 15 2003 10:00:19

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-29.DWG

REFERENCE:
HJW-GeoSpatial, Inc., Upland Topography
NAD27, NGVD29 – CCS Zone III

NOTE:

1. CONFIGURATION OF STONE COLUMNS IS THE SAME AS SHOWN IN DETAIL 4 OF ALTERNATIVE 4 (FIGURE 4-23).

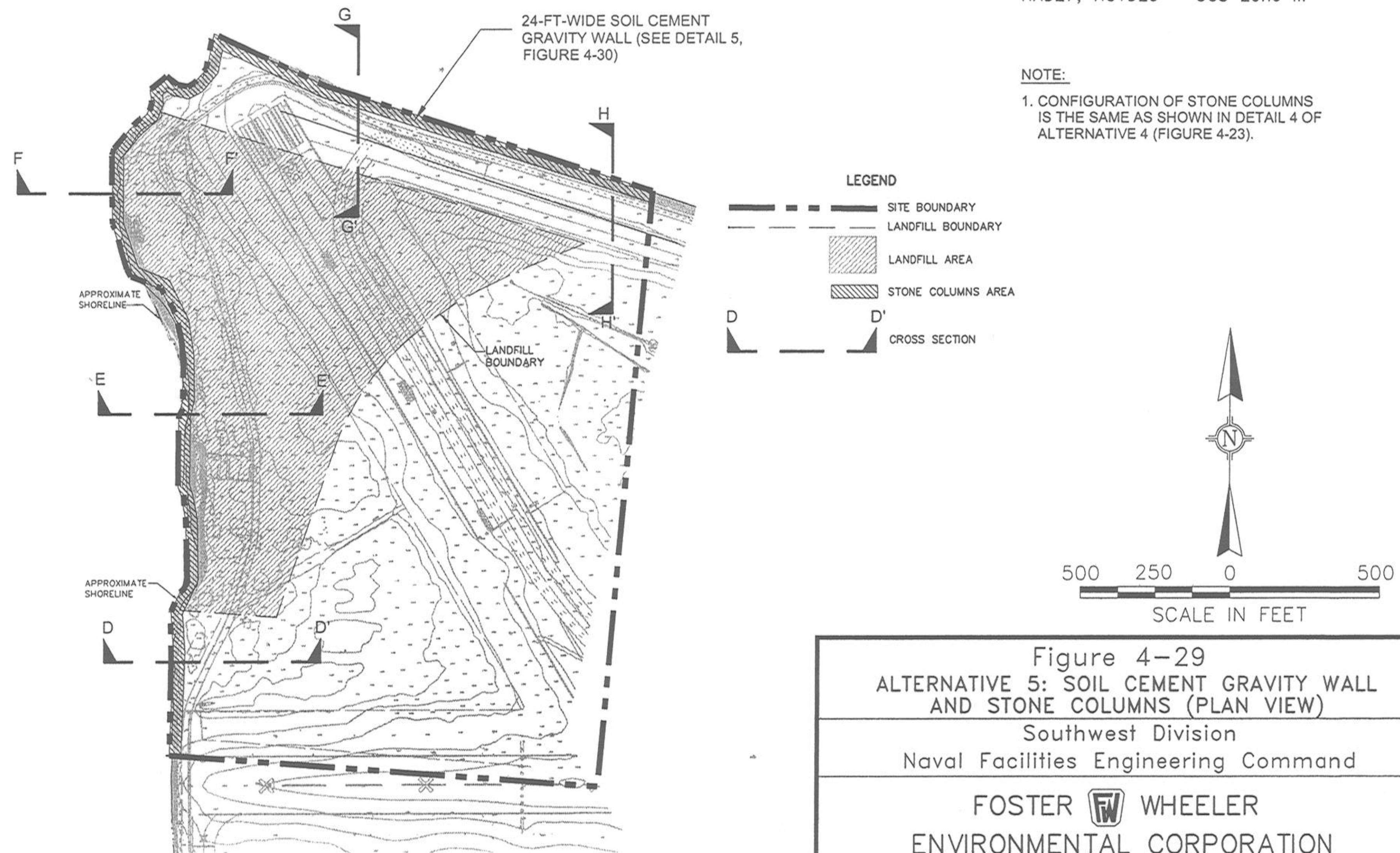


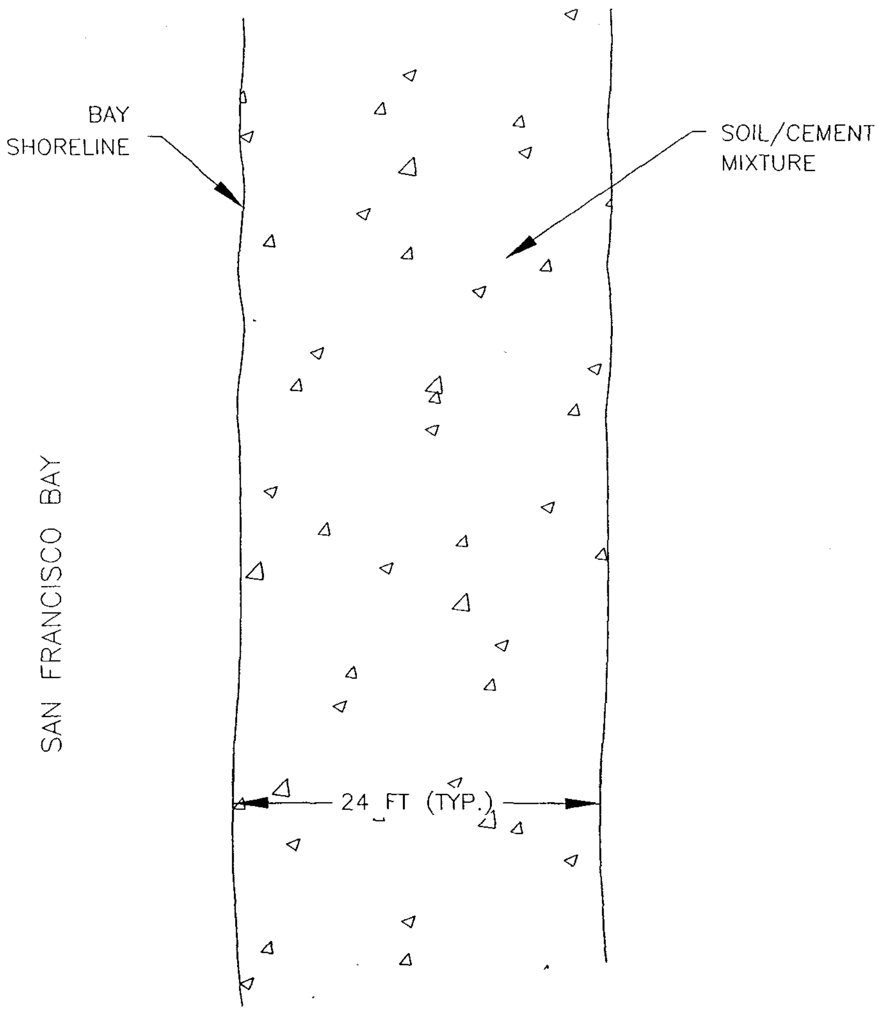
Figure 4–29
ALTERNATIVE 5: SOIL CEMENT GRAVITY WALL AND STONE COLUMNS (PLAN VIEW)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER ENVIRONMENTAL CORPORATION

DETAIL 5

PLAN VIEW



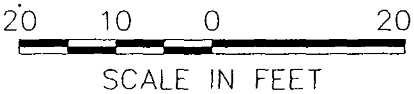


Figure 4-30
ALTERNATIVE 5 – SOIL CEMENT GRAVITY WALL AND STONE COLUMNS (DETAIL 5)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWING NO: 0313684-30.DWG

DCN: FWSD-RAC-03-1368

CTO: # 0054

APPROVED BY: A.L.

CHECKED BY: T.L.

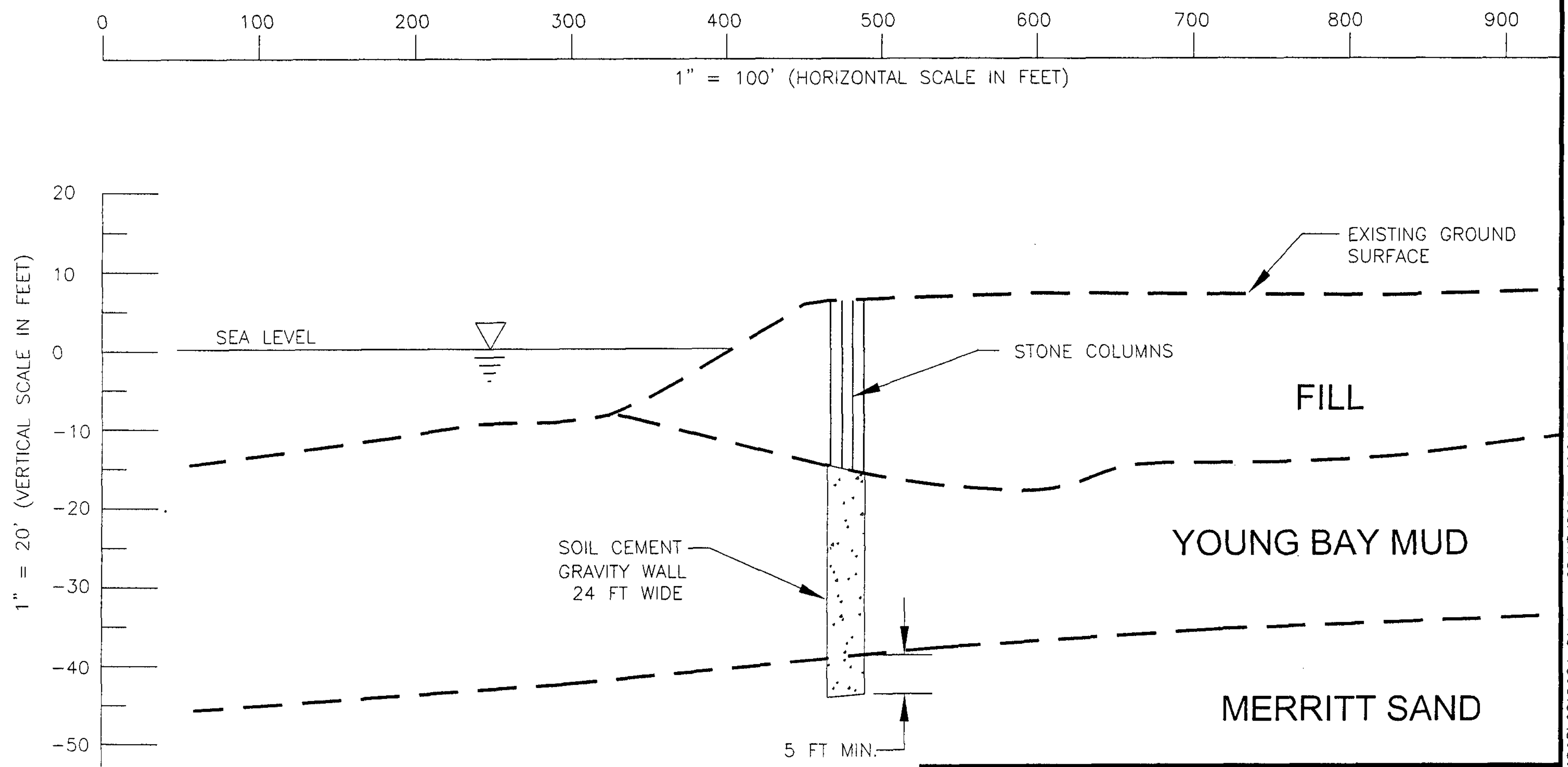
REV: REVISION 0

DRAWN BY: M.D.

DATE: 09/26/03

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-30.dwg 09/15/2003 10:03:02 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-31.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION D-D' (LOOKING NORTH)

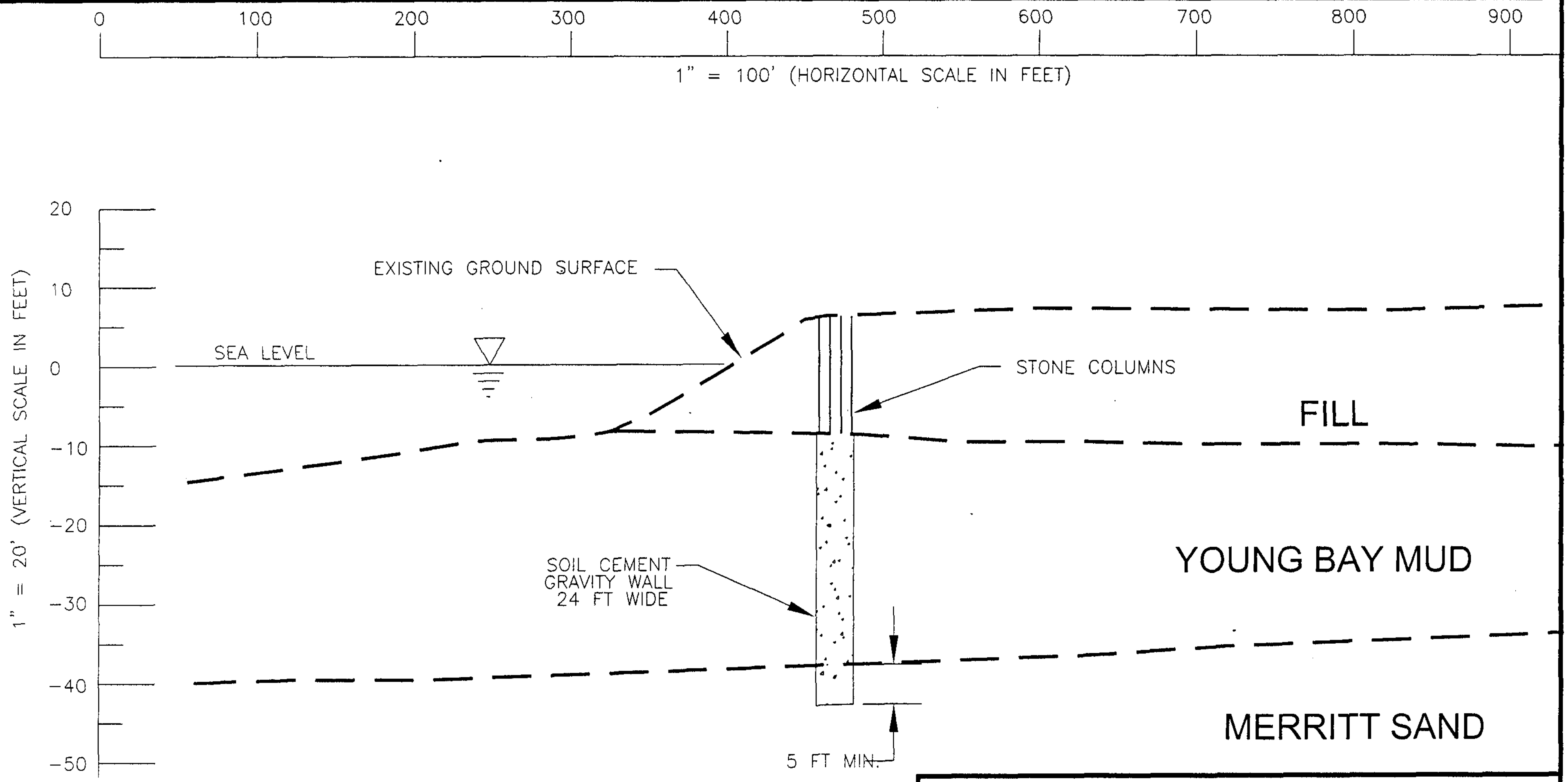
Figure 4-31
ALTERNATIVE 5 – SOIL CEMENT GRAVITY WALL AND STONE COLUMNS (SECTION D-D')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313368\0313684-31.dwg 09/15/2003 10:04:37 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-32.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION E-E' (LOOKING NORTH)

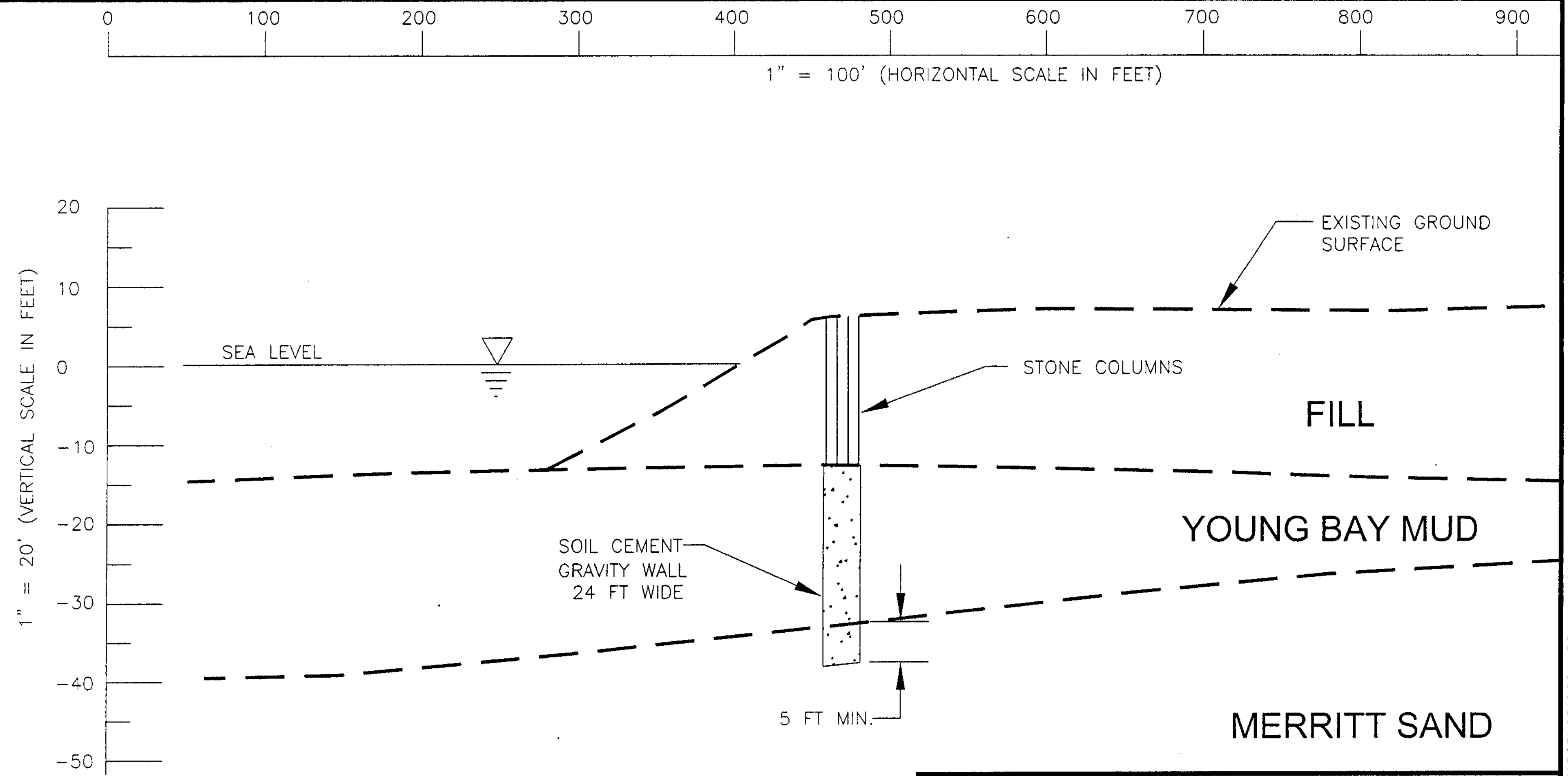
Figure 4-32
ALTERNATIVE 5 – SOIL CEMENT GRAVITY WALL AND STONE COLUMNS (SECTION E-E')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-32.dwg 09/15/2003 10:07:03 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-33.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION F–F' (LOOKING NORTH)

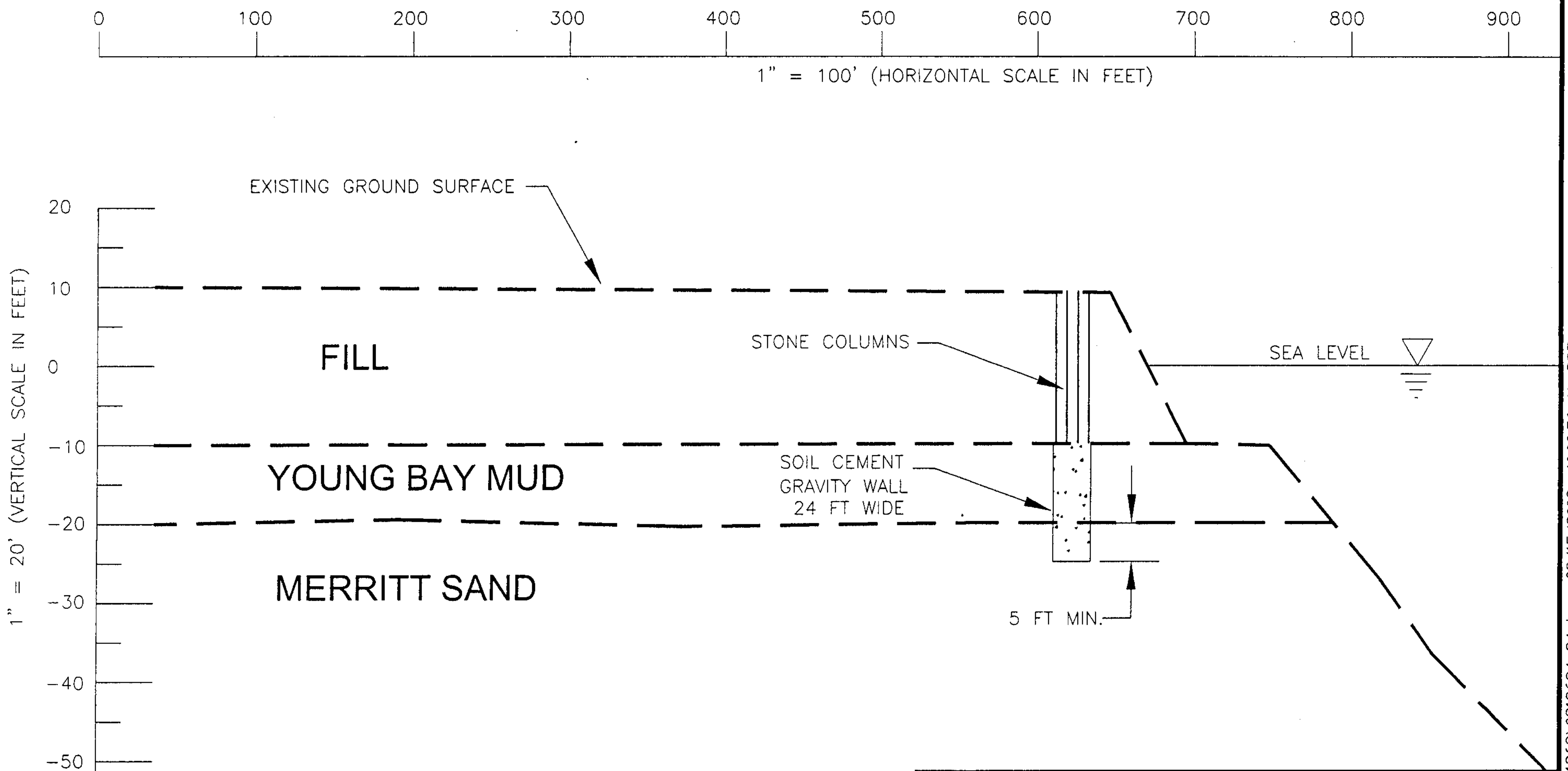
Figure 4–33
ALTERNATIVE 5 – SOIL CEMENT GRAVITY WALL AND STONE COLUMNS (SECTION F–F')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-33.dwg 09/15/2003 10:07:48 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-34.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION G-G' (LOOKING WEST)

Figure 4-34
ALTERNATIVE 5 - SOIL CEMENT GRAVITY WALL AND STONE COLUMNS (SECTION G-G')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-34.dwg 09/15/2003 10:09:56 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313484-35.DWG

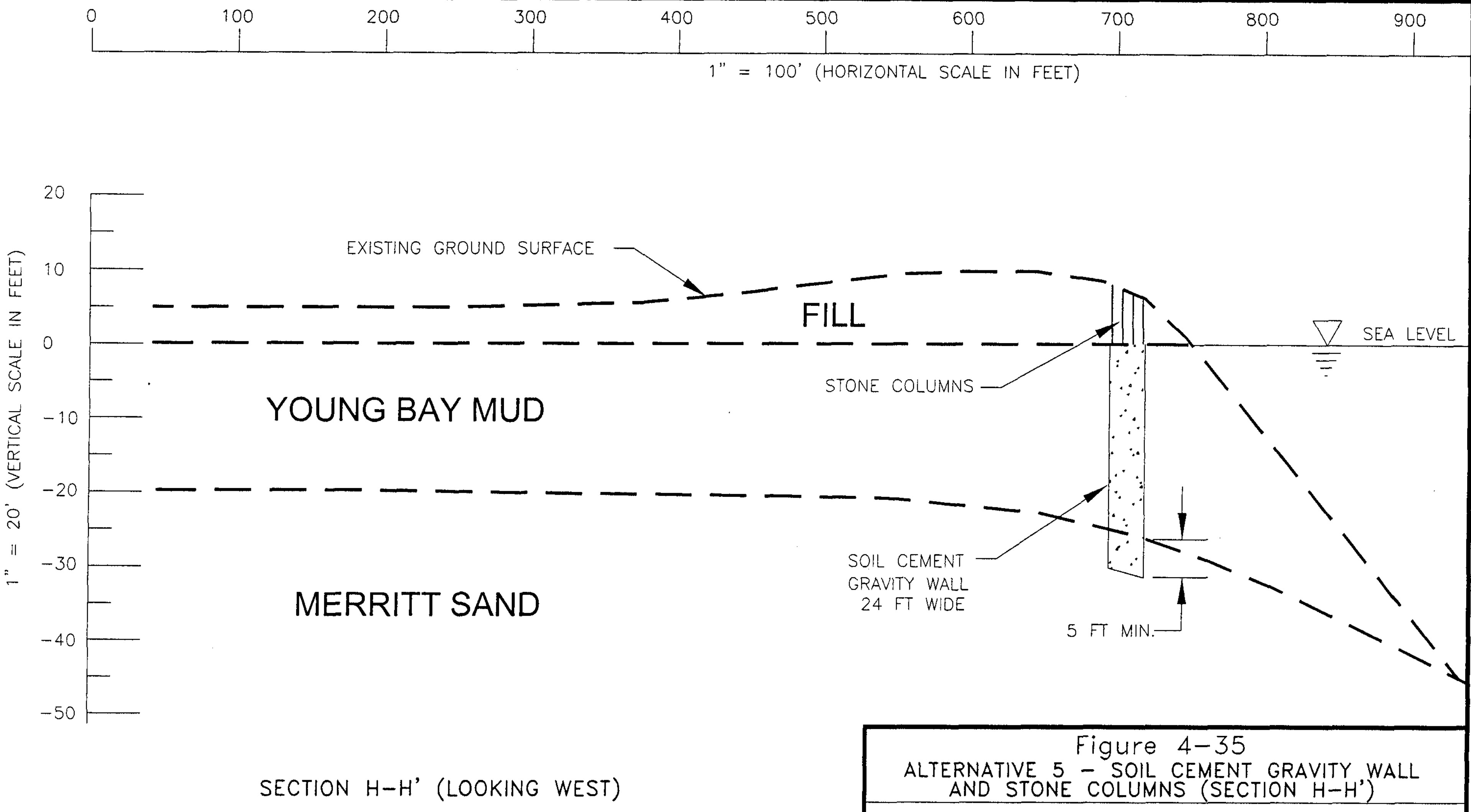


Figure 4-35
ALTERNATIVE 5 - SOIL CEMENT GRAVITY WALL AND STONE COLUMNS (SECTION H-H')

Southwest Division
Naval Facilities Engineering Command

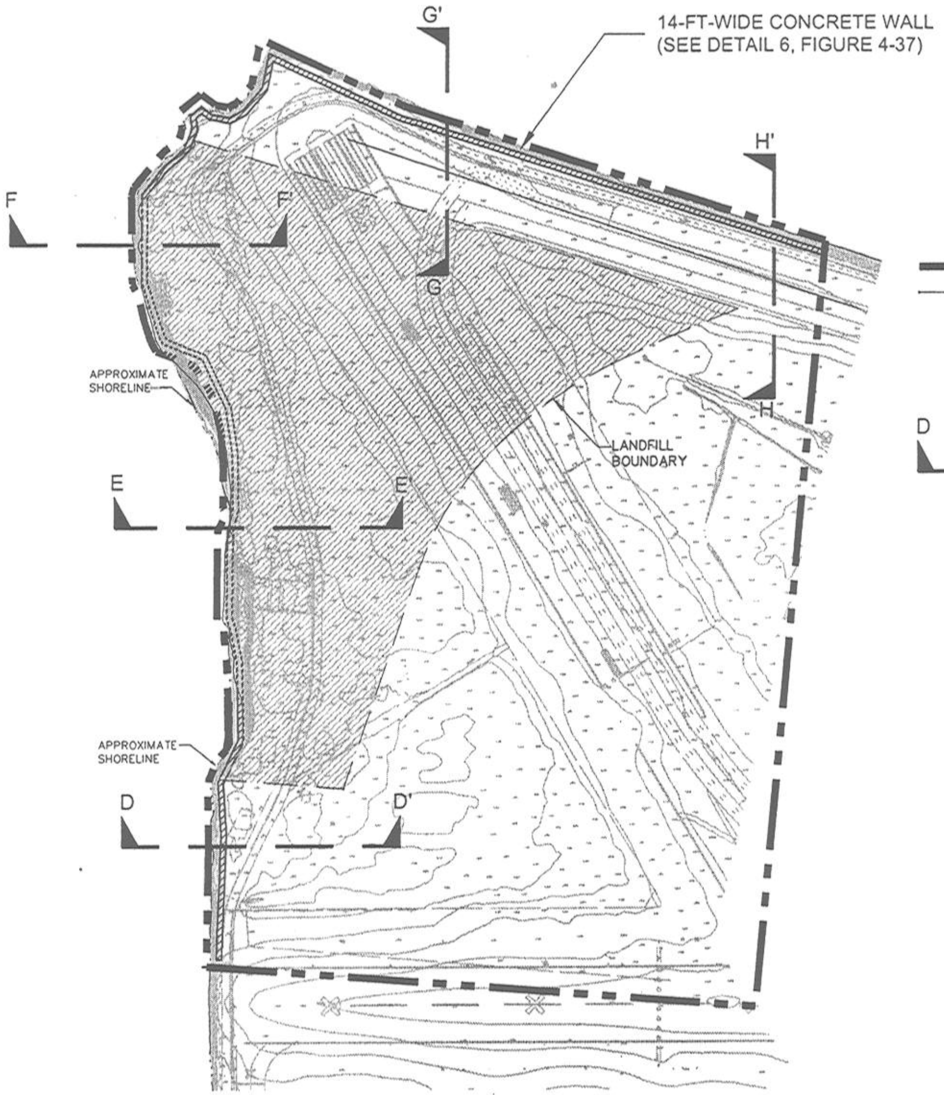
FOSTER WHEELER
ENVIRONMENTAL CORPORATION

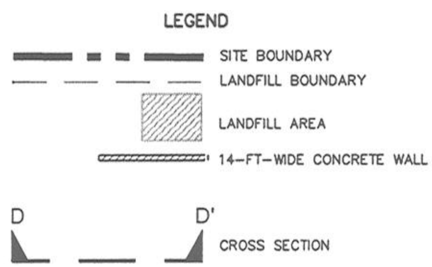
I:\1990-RAC\Cto-0054\Dwg\031368\0313684-35.dwg 09/15/2003 10:10:28 AM PDT

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-36.DWG

I:\1990-RAC\CTO-0054\DWG\031368\0313684-36.DWG
PLOT/UPDATE: SEP 15 2003 10:11:45

REFERENCE:
HJW-GeoSpatial, Inc., Upland Topography
NAD27, NGVD29 - CCS Zone III





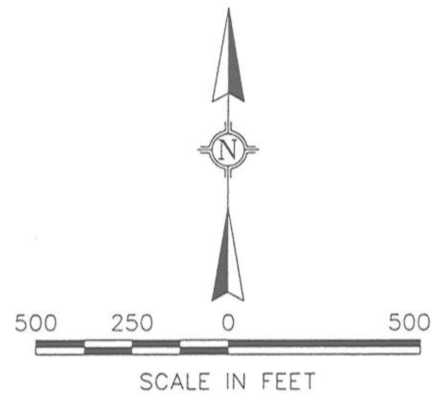


Figure 4-36
ALTERNATIVE 6 - CONCRETE WALL
(PLAN VIEW)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DETAIL 6

PLAN VIEW

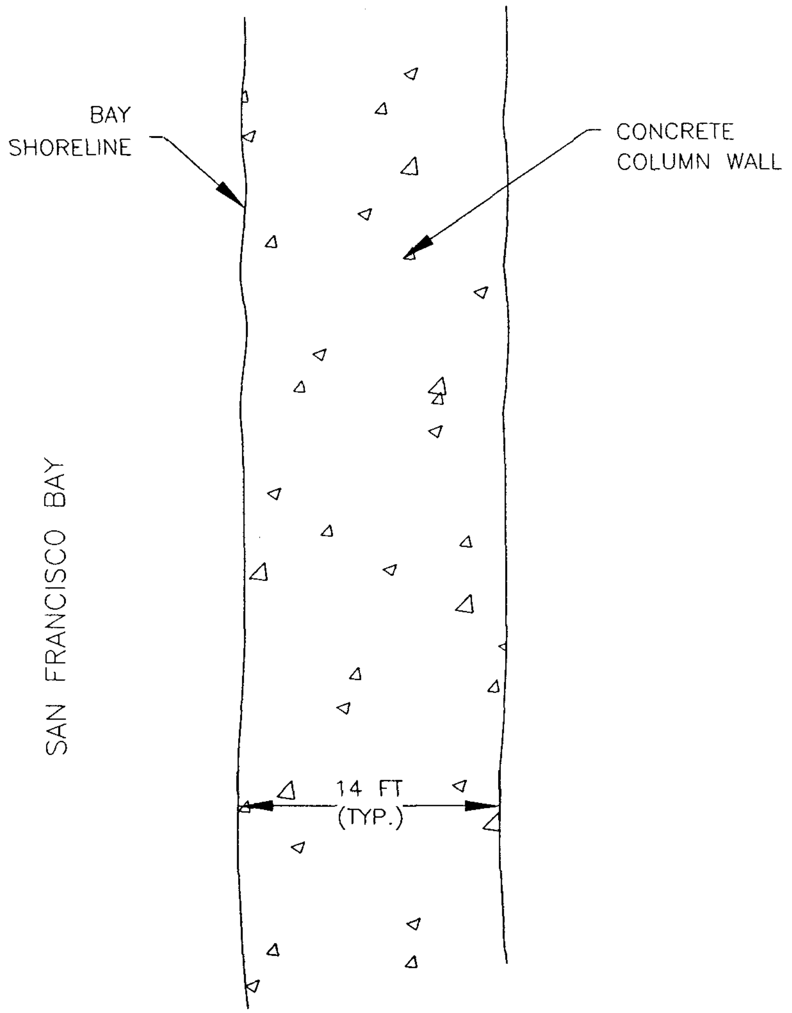


Figure 4-37

ALTERNATIVE 6 – CONCRETE WALL (DETAIL 6)

Southwest Division

Naval Facilities Engineering Command

FOSTER WHEELER ENVIRONMENTAL CORPORATION

DRAWING NO: 0313684-37.DWG

DCN: FWSD-RAC-03-1368

CTO: # 0054

APPROVED BY: A.L.

CHECKED BY: T.L.

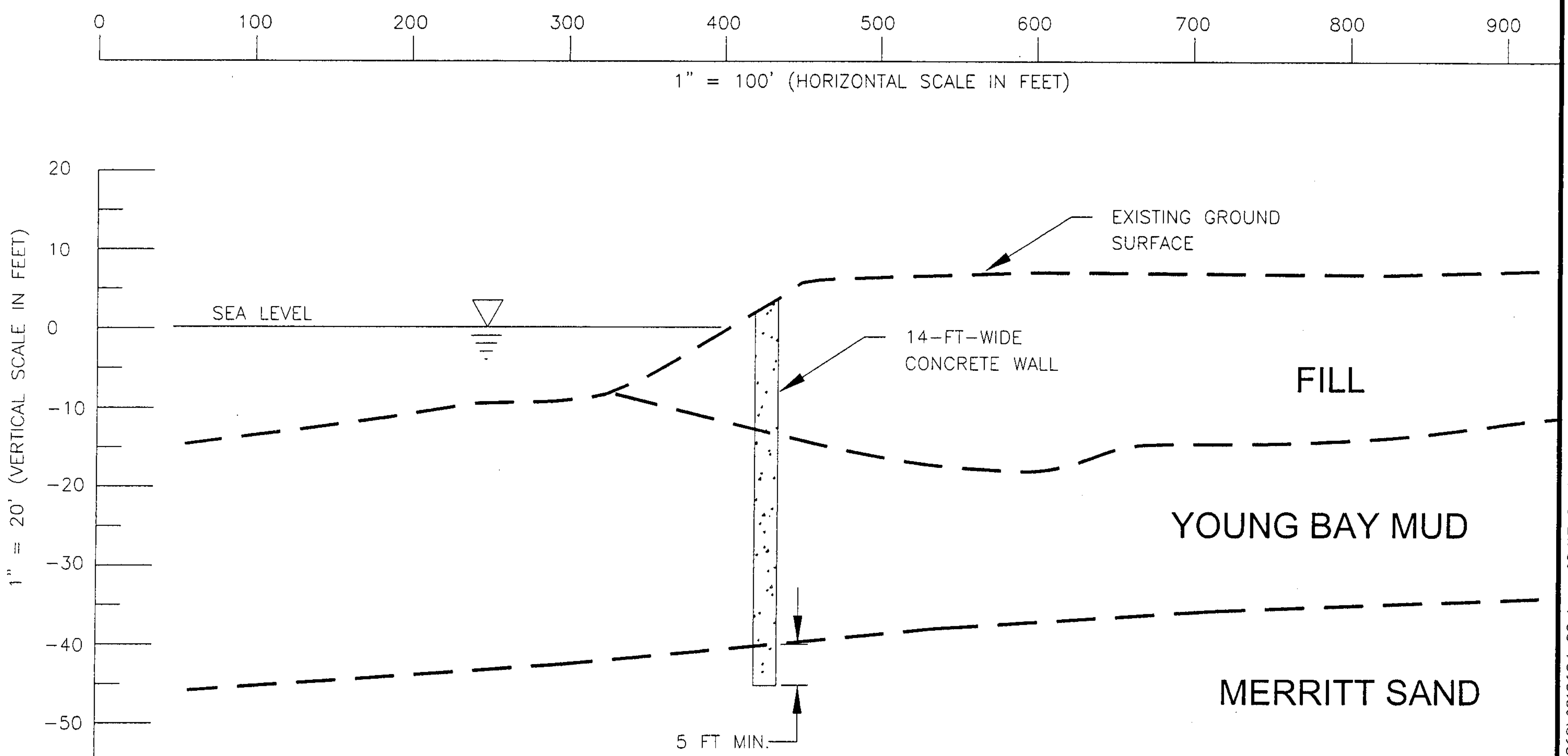
REV: REVISION 0

DRAWN BY: M.D.

DATE: 09/26/03

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-37.dwg 09/15/2003 10:12:46 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-38.DWG



SECTION D-D' (LOOKING NORTH)

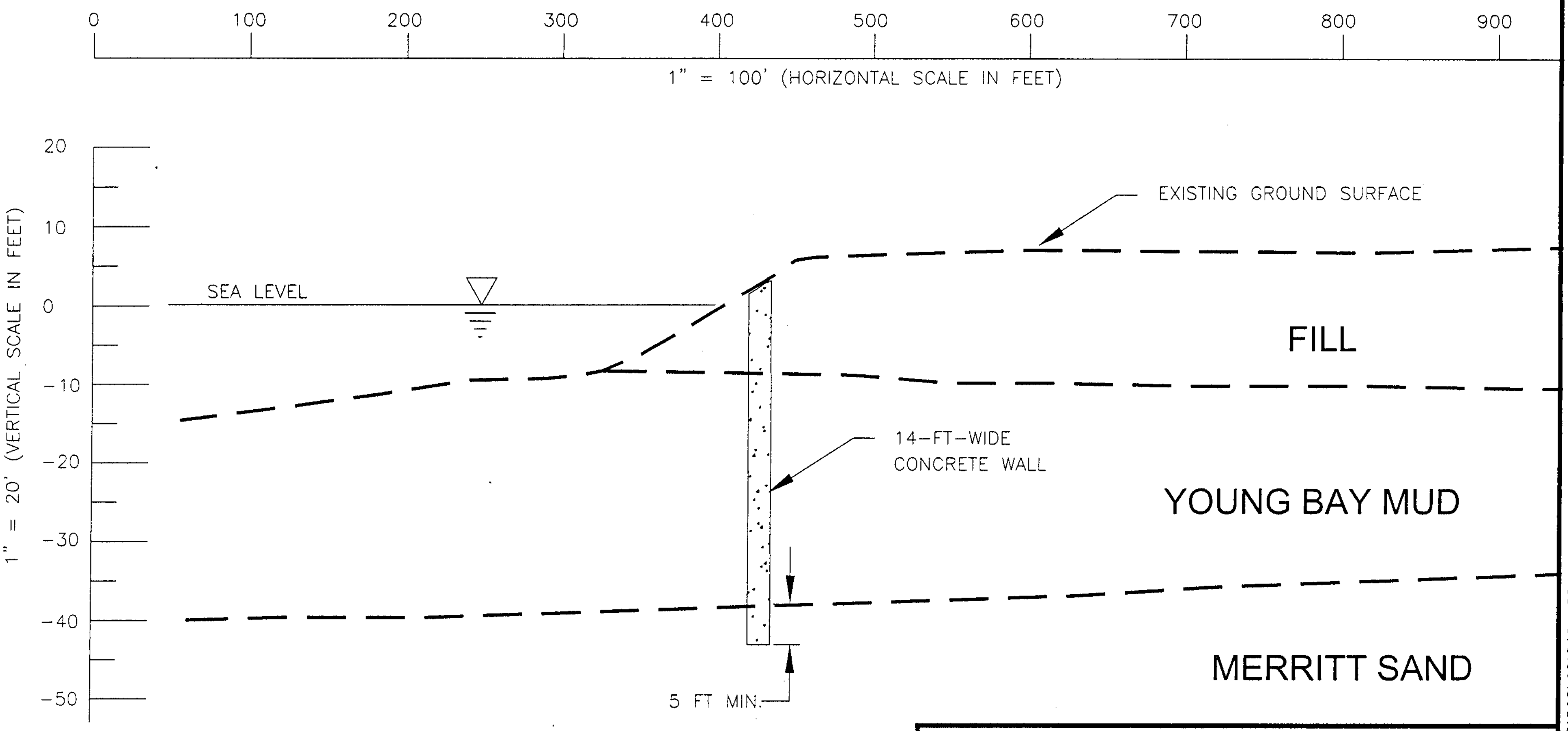
Figure 4-38
ALTERNATIVE 6 – CONCRETE WALL
(SECTION D-D')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-38.dwg 09/15/2003 10:14:01 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-39.DWG



SECTION E-E' (LOOKING NORTH)

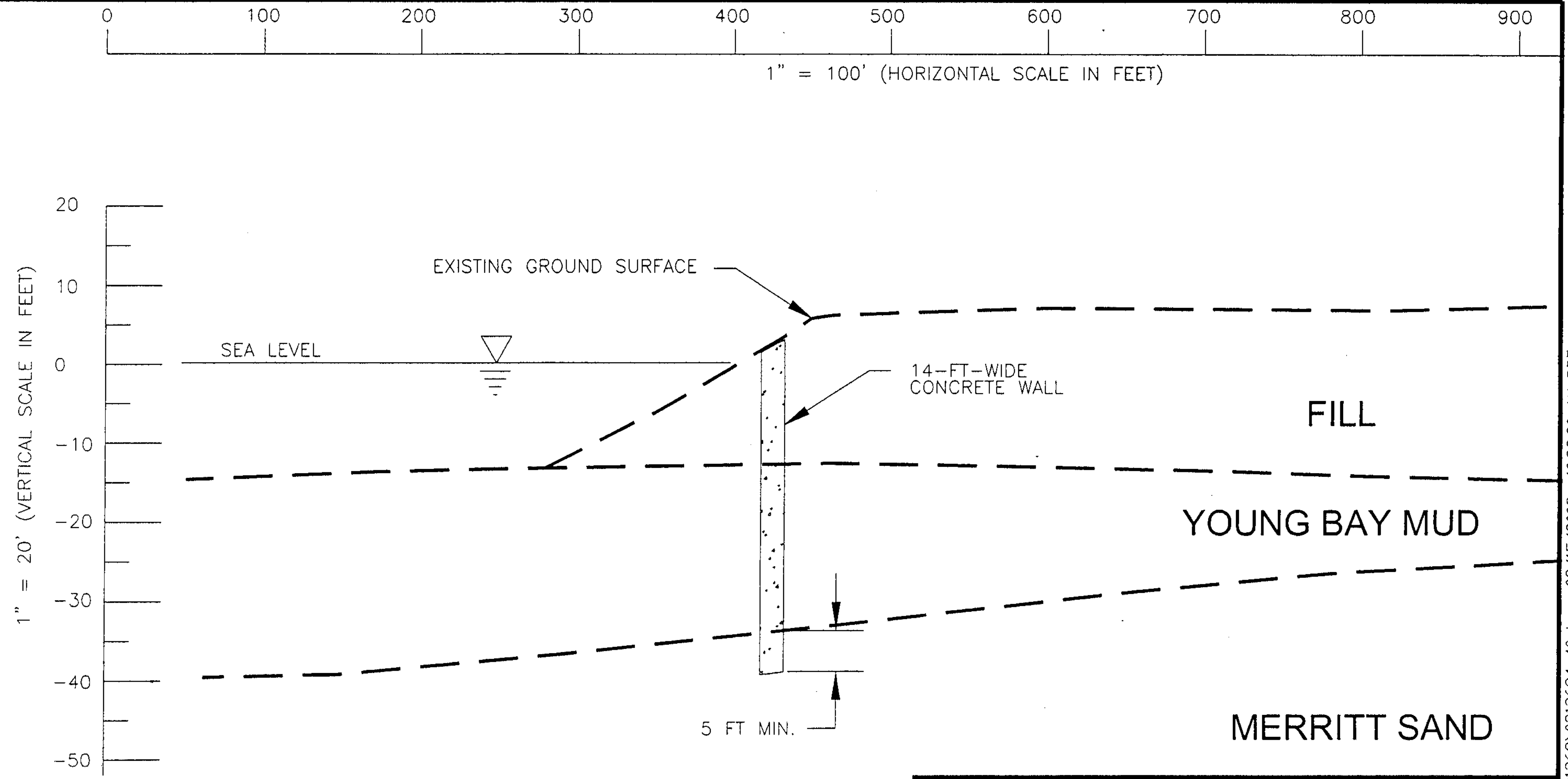
Figure 4-39
ALTERNATIVE 6 - CONCRETE WALL
(SECTION E-E')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-39.dwg 09/15/2003 10:24:43 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-40.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION F-F' (LOOKING NORTH)

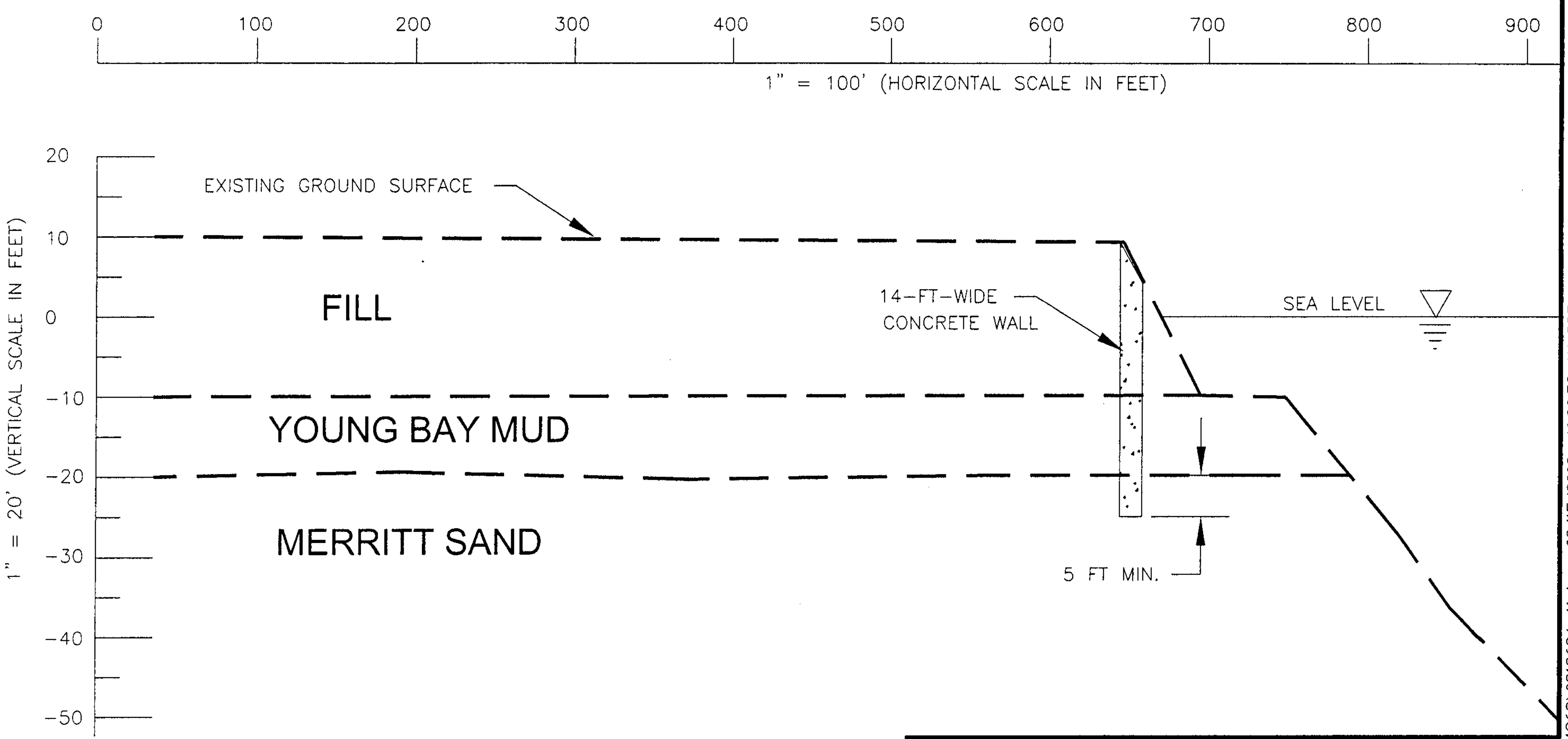
Figure 4-40
ALTERNATIVE 6 - CONCRETE WALL
(SECTION F-F')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-40.dwg 09/15/2003 10:28:20 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-41.DWG



SECTION G-G' (LOOKING WEST)

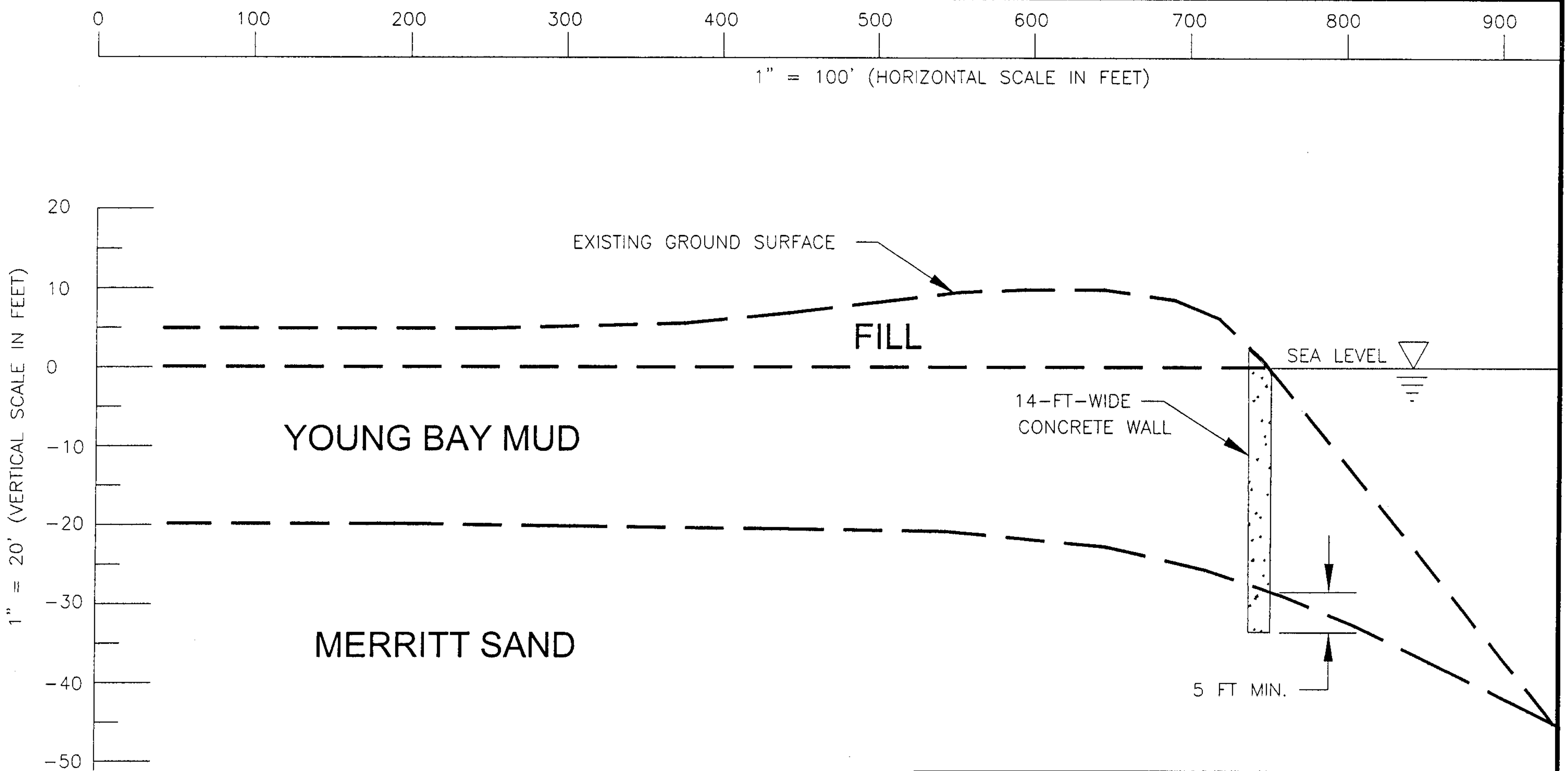
Figure 4-41
ALTERNATIVE 6 – CONCRETE WALL
(SECTION G-G')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-41.dwg 09/15/2003 10:29:58 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RACII-03-1368	DRAWING NO: 0313684-42.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: #0054	



SECTION H-H' (LOOKING WEST)

Figure 4-42
ALTERNATIVE 6 – CONCRETE WALL
(SECTION H-H')

Southwest Division
Naval Facilities Engineering Command

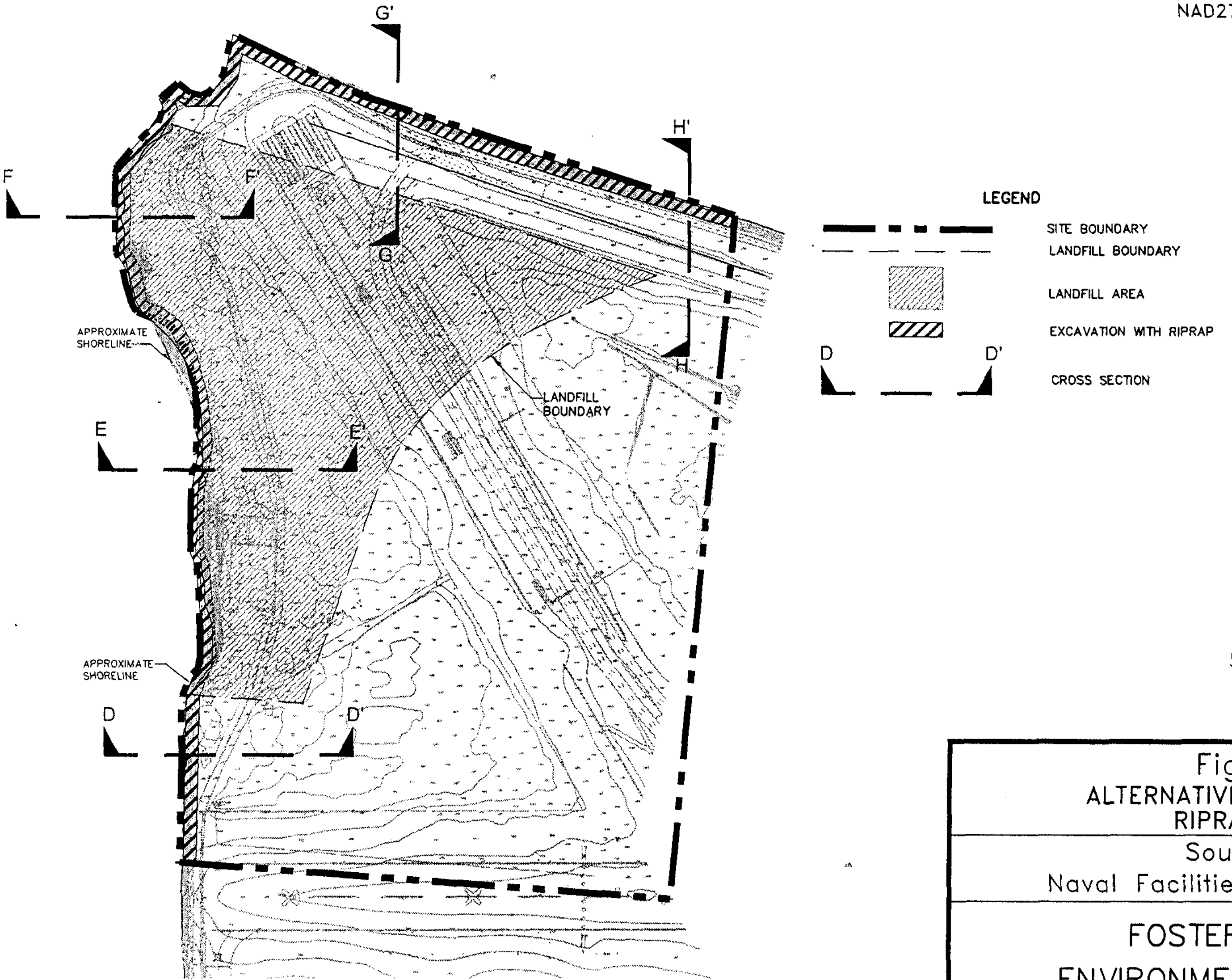
FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-42.dwg 09/15/2003 10:30:48 AM PDT

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-43.DWG

I:\1990-RAC\CTO-0054\DWG\031368\0313684-43.DWG
PLOT/UPDATE: SEP 15 2003 10:32:54

REFERENCE:
HJW-GeoSpatial, Inc., Upland Topography
NAD27, NGVD29 - CCS Zone III



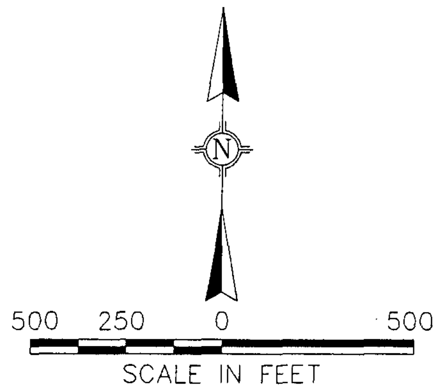
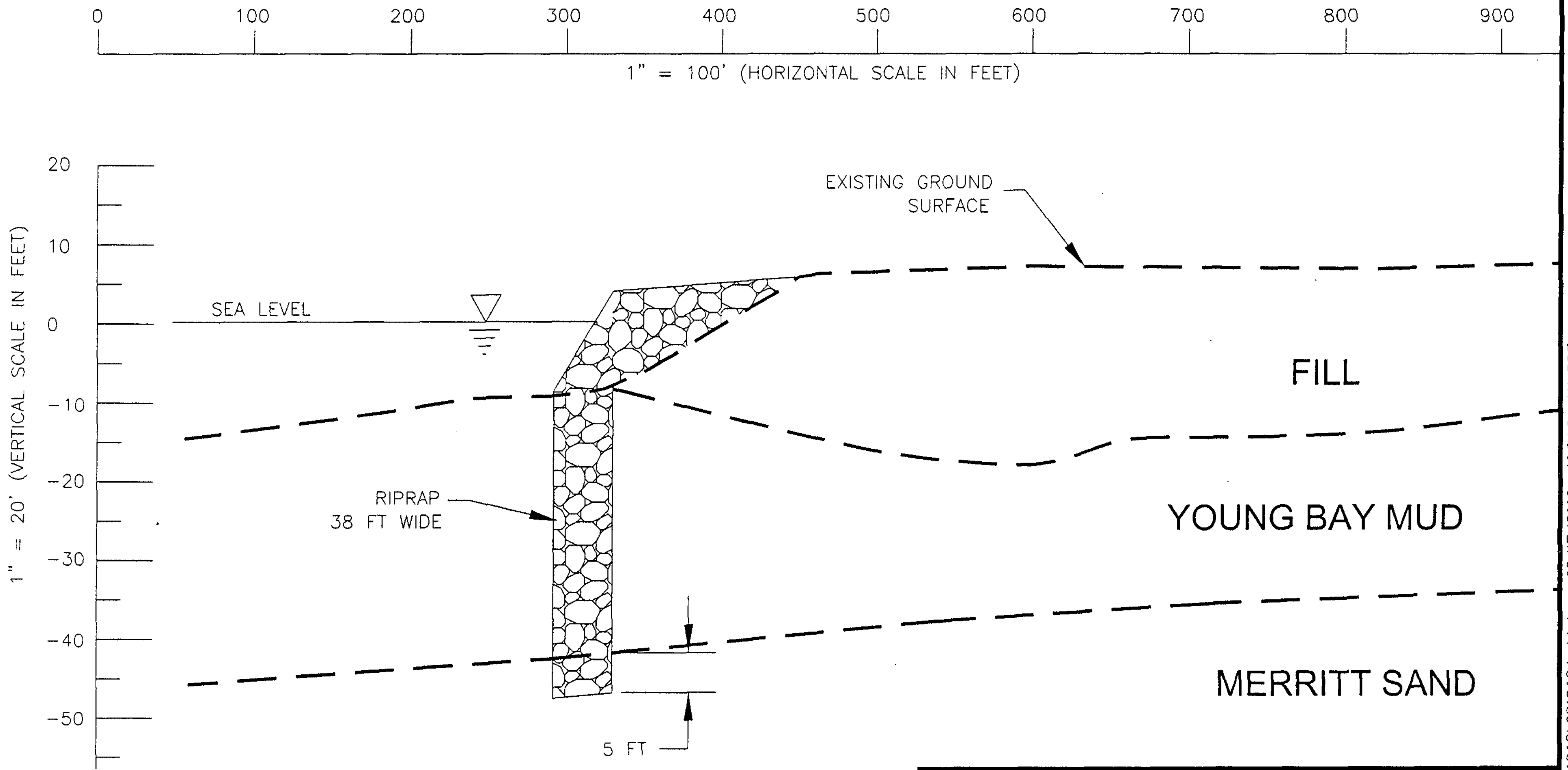


Figure 4-43
ALTERNATIVE 7: EXCAVATION WITH RIPRAP (PLAN VIEW)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313484-44.DWG



SECTION D-D' (LOOKING NORTH)

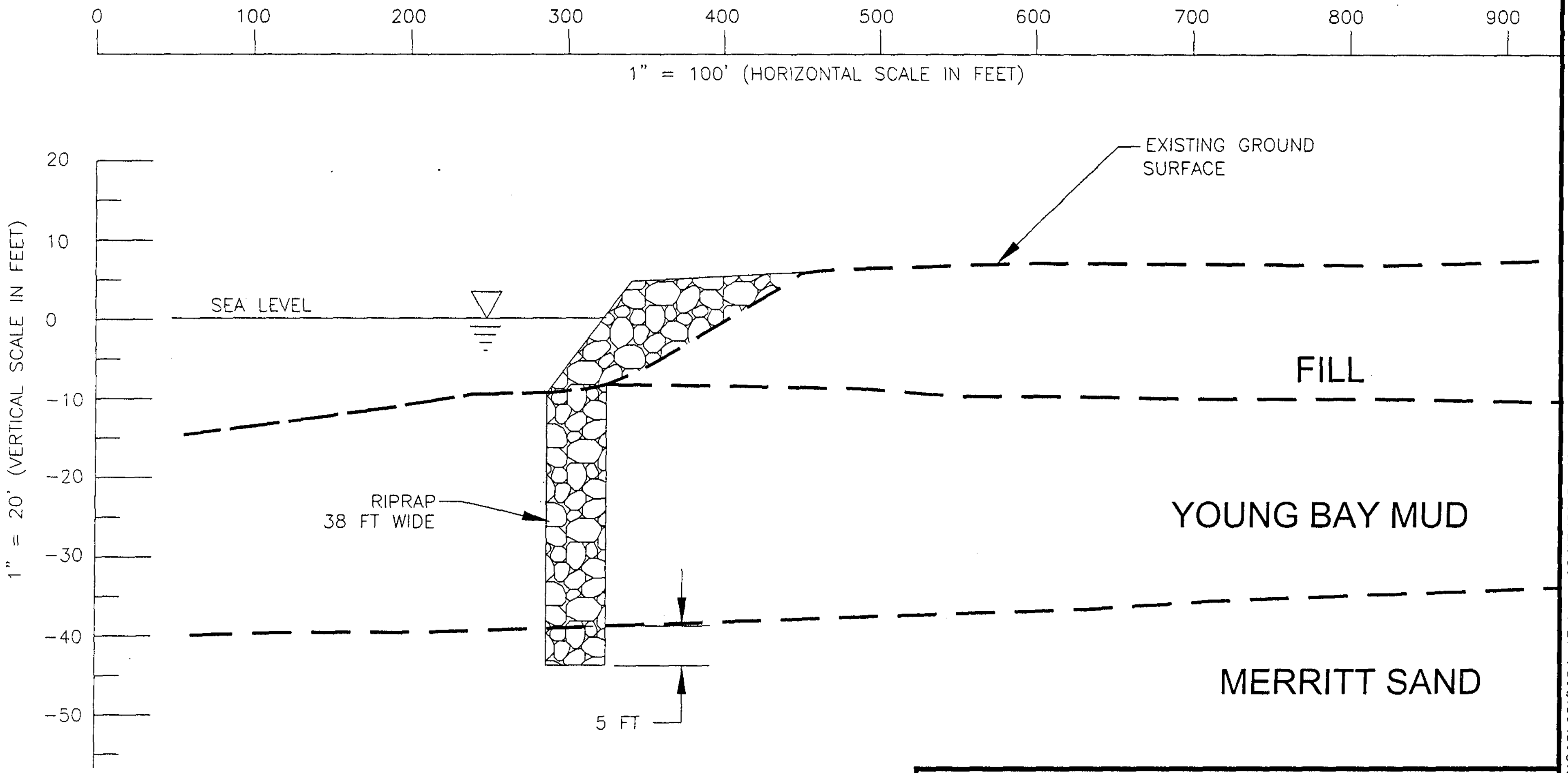
Figure 4-44
ALTERNATIVE 7 - EXCAVATION WITH RIPRAP (SECTION D-D')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313368\0313684-44.dwg 09/15/2003 10:34:31 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-45.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION E-E' (LOOKING NORTH)

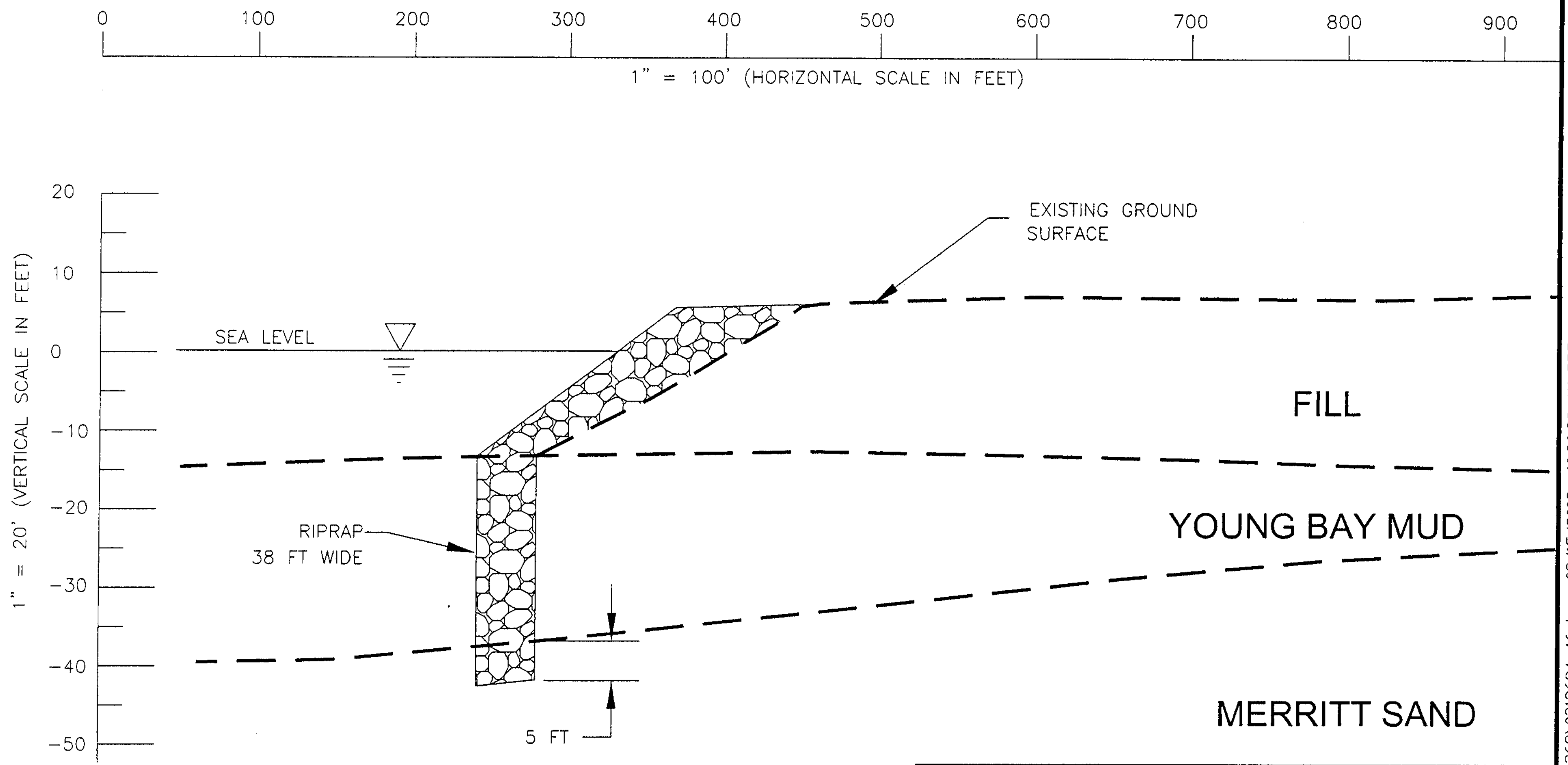
Figure 4-45
ALTERNATIVE 7 - EXCAVATION WITH RIPRAP (SECTION E-E')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-45.dwg 09/15/2003 10:35:16 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-46.DWG



SECTION F-F' (LOOKING NORTH)

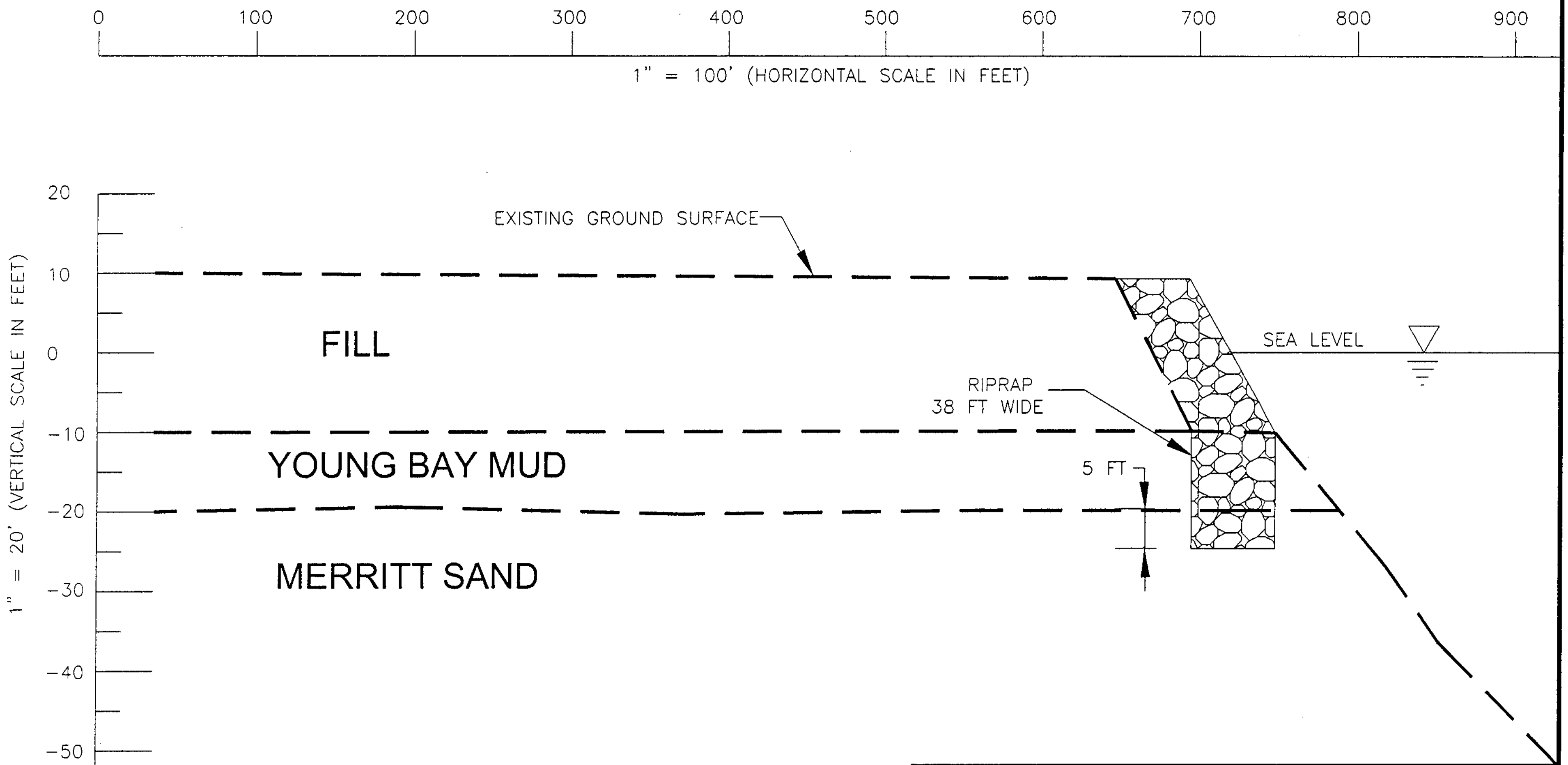
Figure 4-46
ALTERNATIVE 7 - EXCAVATION WITH RIPRAP (SECTION F-F')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-46.dwg 09/15/2003 10:36:23 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-47.DWG



SECTION G-G' (LOOKING WEST)

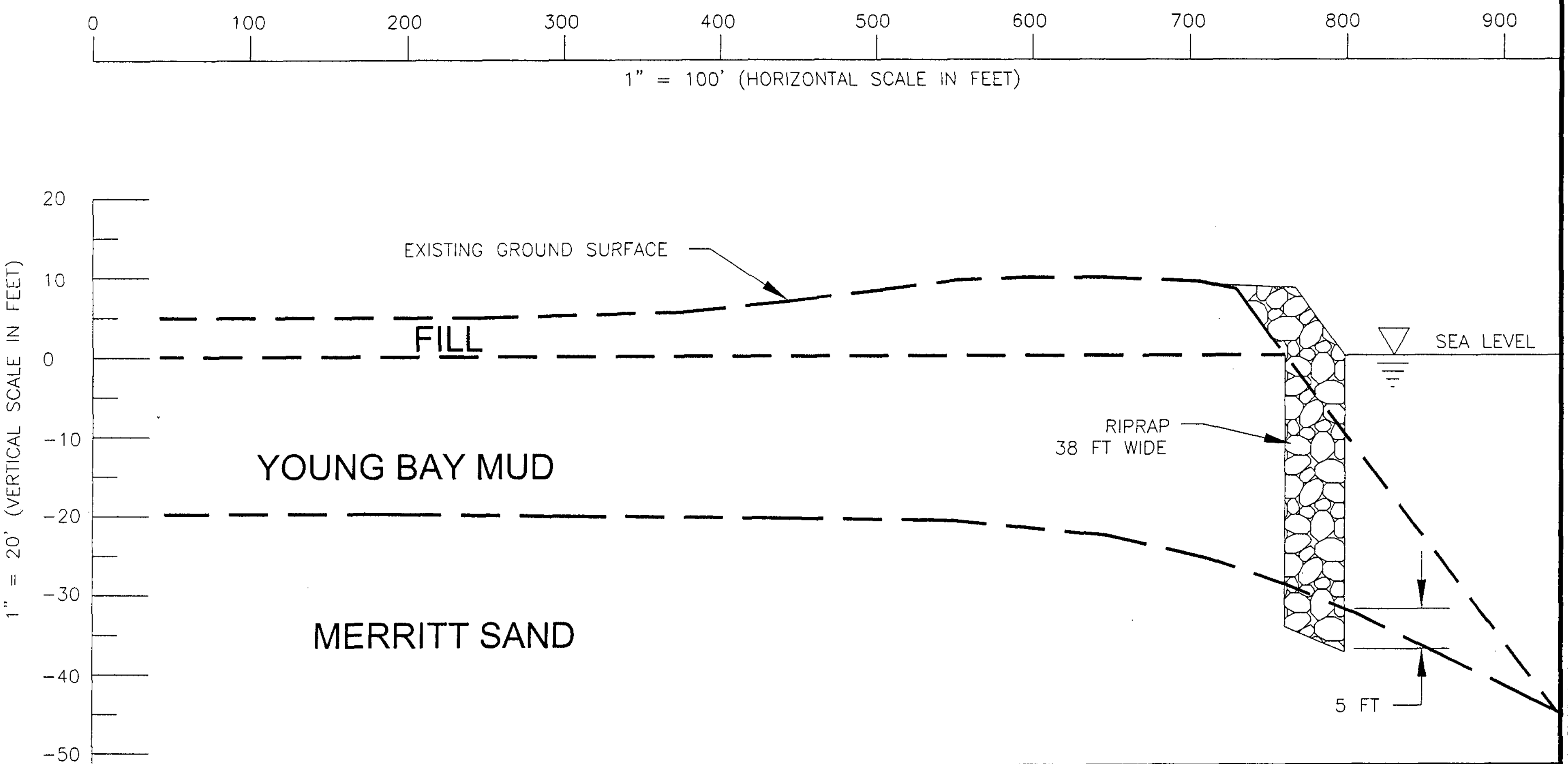
Figure 4-47
ALTERNATIVE 7 - EXCAVATION WITH RIPRAP (SECTION G-G')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-47.dwg 09/15/2003 10:37:10 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-48.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION H-H' (LOOKING WEST)

Figure 4-48
ALTERNATIVE 7 - EXCAVATION WITH
RIPRAP (SECTION H-H')

Southwest Division
Naval Facilities Engineering Command

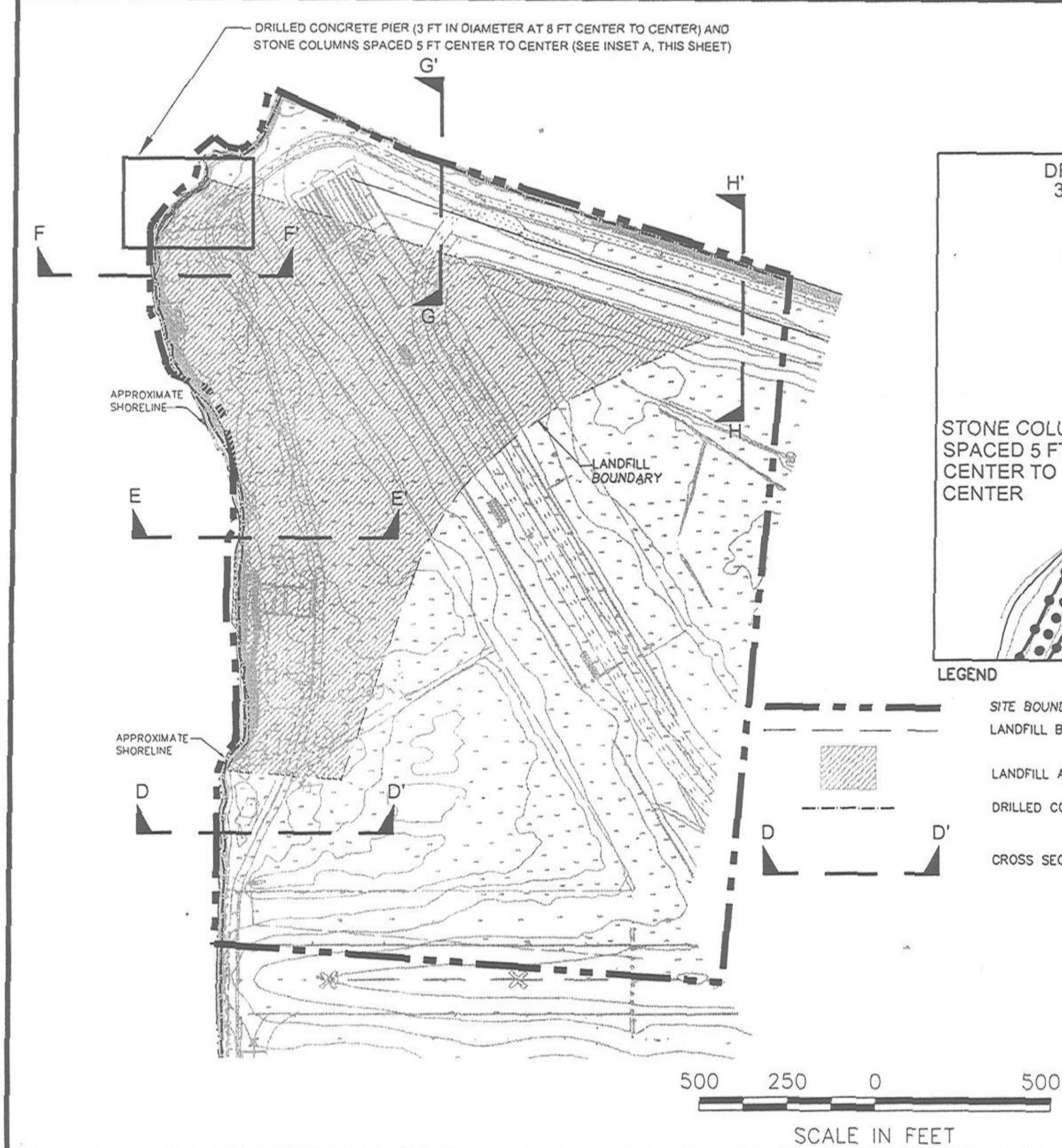
FOSTER WHEELER
ENVIRONMENTAL CORPORATION

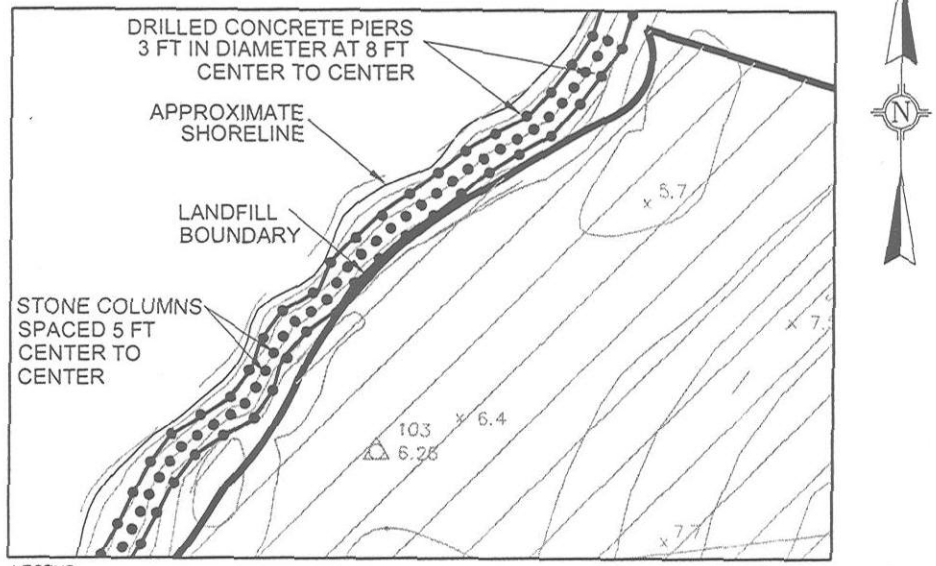
I:\1990-RAC\Cto-0054\Dwg\031368\0313684-48.dwg 09/15/2003 10:38:28 AM PDT

J:\1990-RAC\CTO-0054\DWG\031368\0313684-49.DWG
PLOT/UPDATE: SEP 15 2003 10:39:59

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-49.DWG

REFERENCE:
HJW-GeoSpatial, Inc., Upland Topography
NAD27, NGVD29 - CCS Zone III





INSET - A

50 25 0 50
SCALE IN FEET

LEGEND

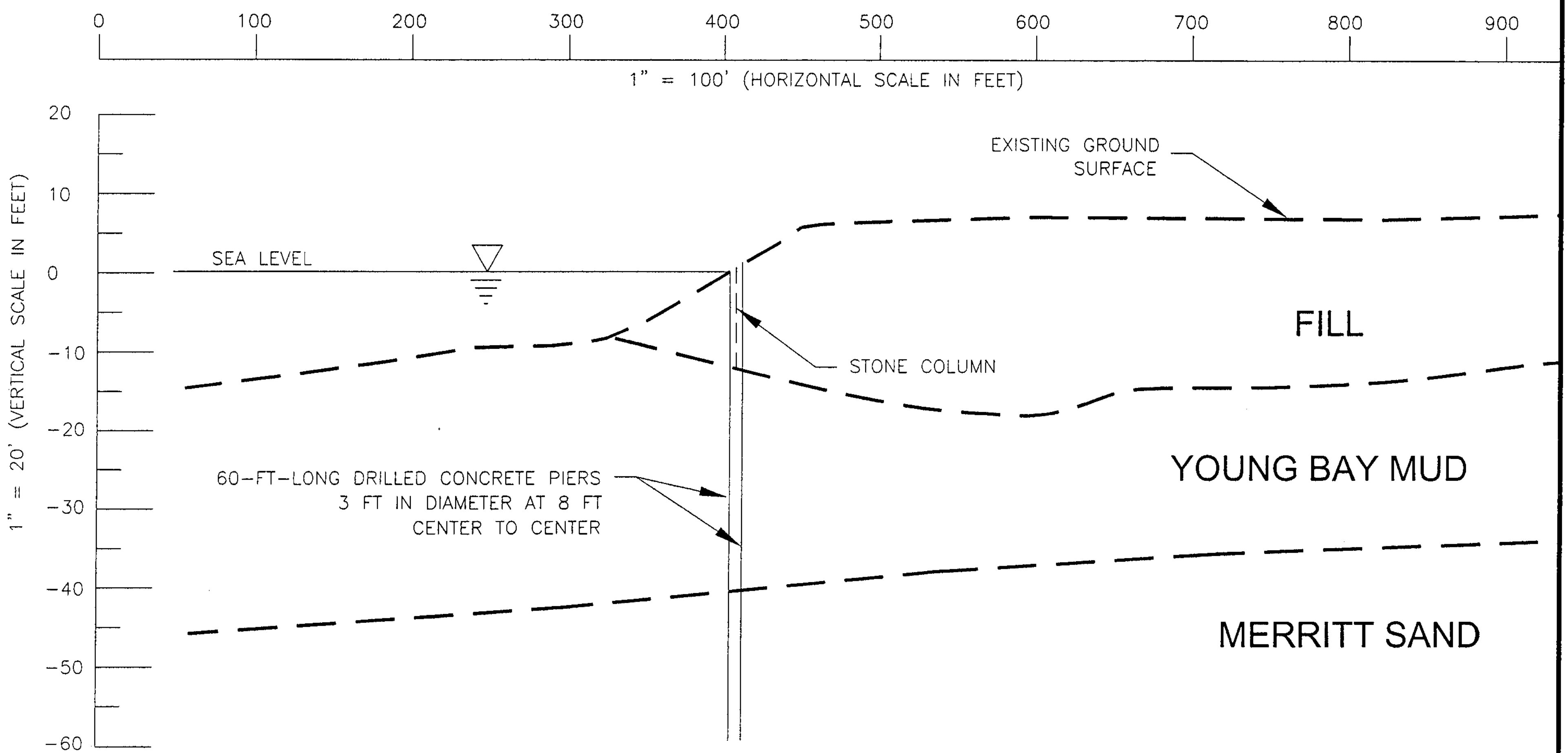
SITE BOUNDARY
LANDFILL BOUNDARY
LANDFILL AREA
DRILLED CONCRETE PIER
D D'
CROSS SECTION

Figure 4-49
ALTERNATIVE 8: DRILLED CONCRETE PIERS WITH STONE COLUMNS (PLAN VIEW)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313484-50.DWG



SECTION D-D' (LOOKING NORTH)

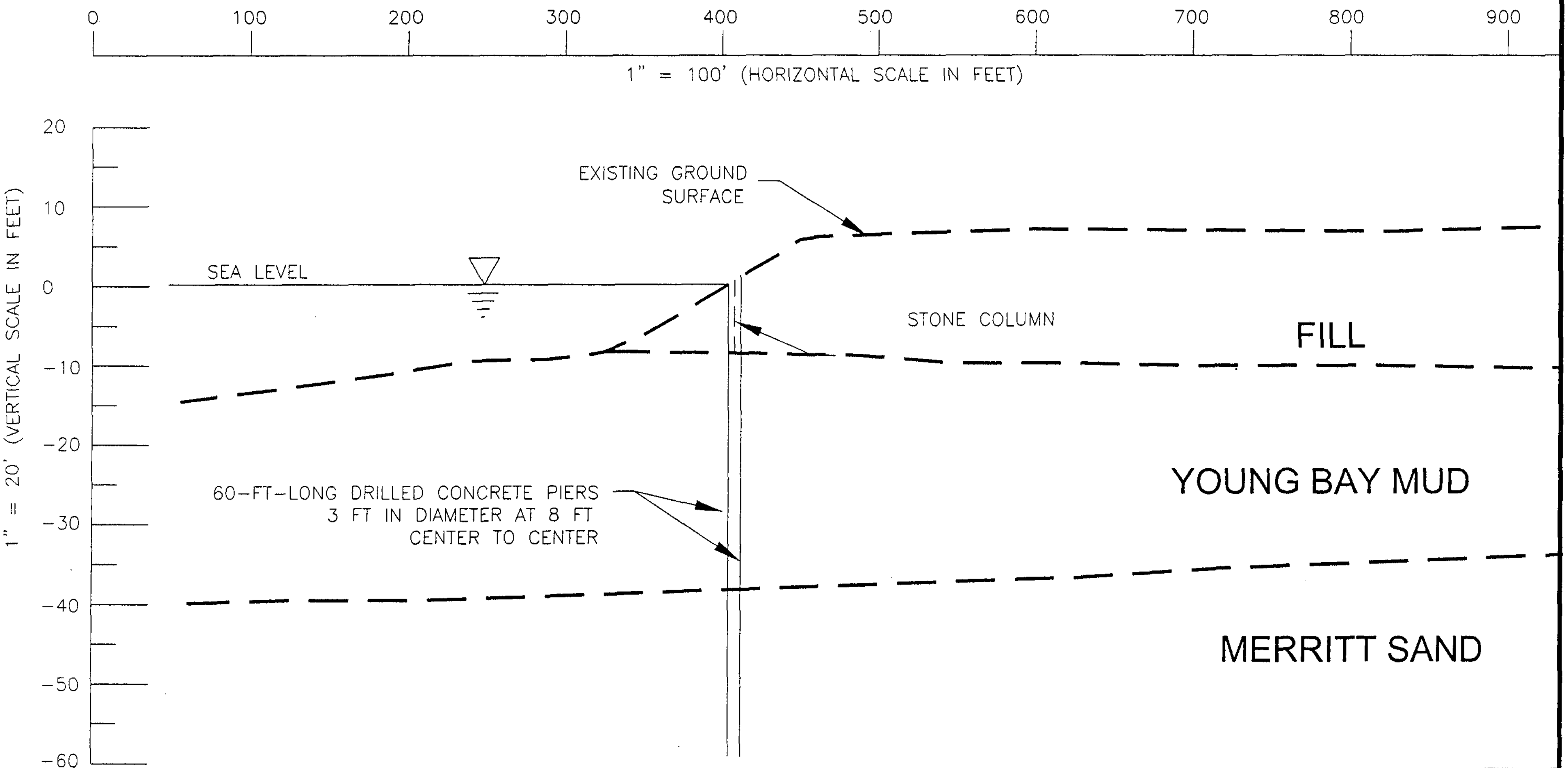
Figure 4-50
ALTERNATIVE 8 – DRILLED CONCRETE PIERS WITH STONE COLUMNS (SECTION D-D')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-50.dwg 09/15/2003 10:42:19 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RACII-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-51.DWG



SECTION E-E' (LOOKING NORTH)

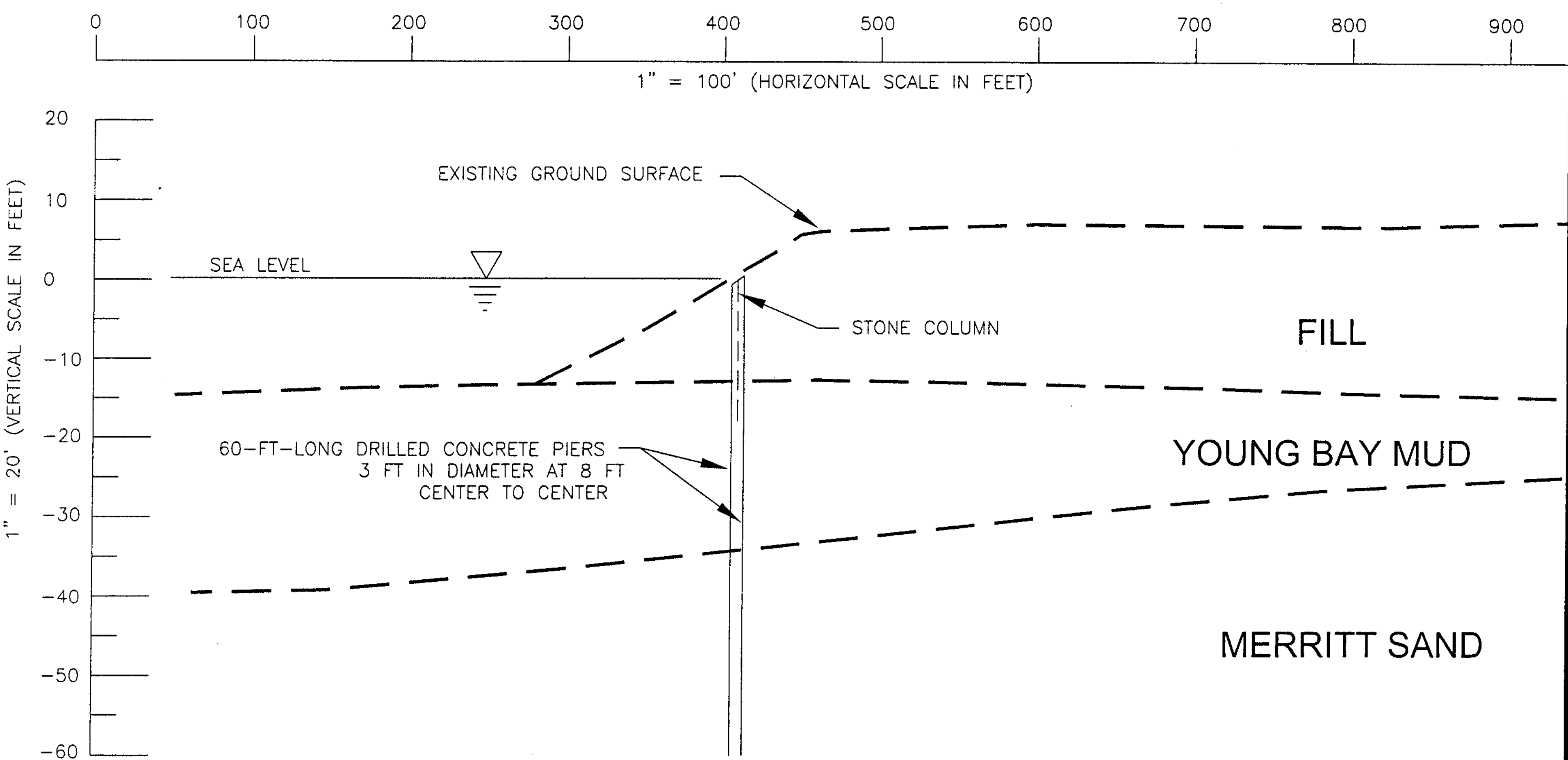
Figure 4-51
ALTERNATIVE 8 – DRILLED CONCRETE PIERS WITH STONE COLUMNS (SECTION E-E')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-51.dwg 09/15/2003 10:44:57 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-52.DWG



SECTION F–F' (LOOKING NORTH)

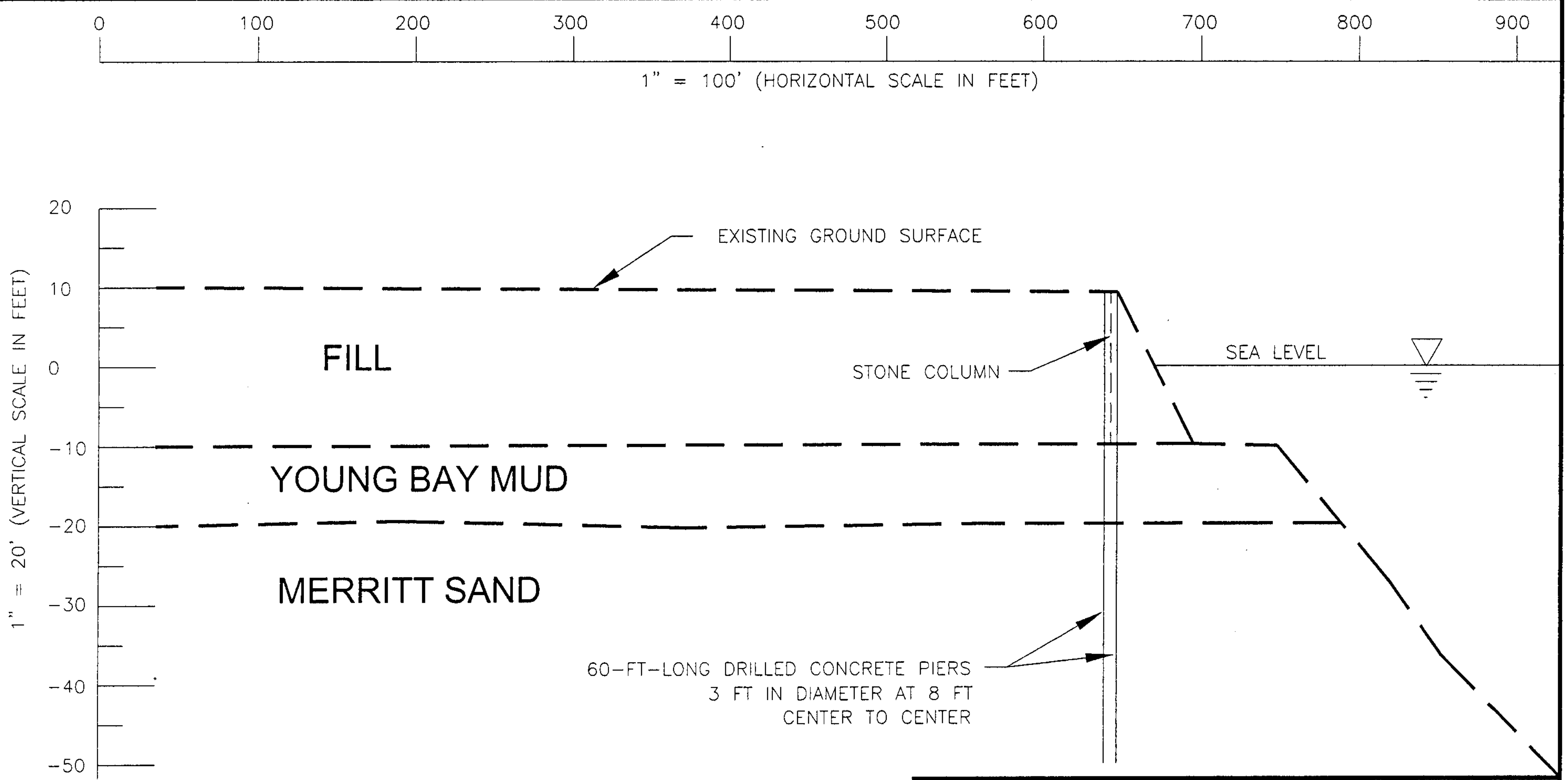
Figure 4–52
ALTERNATIVE 8 – DRILLED CONCRETE PIERS WITH STONE COLUMNS (SECTION F–F')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-52.dwg 09/15/2003 10:45:56 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-53.DWG



SECTION G-G' (LOOKING WEST)

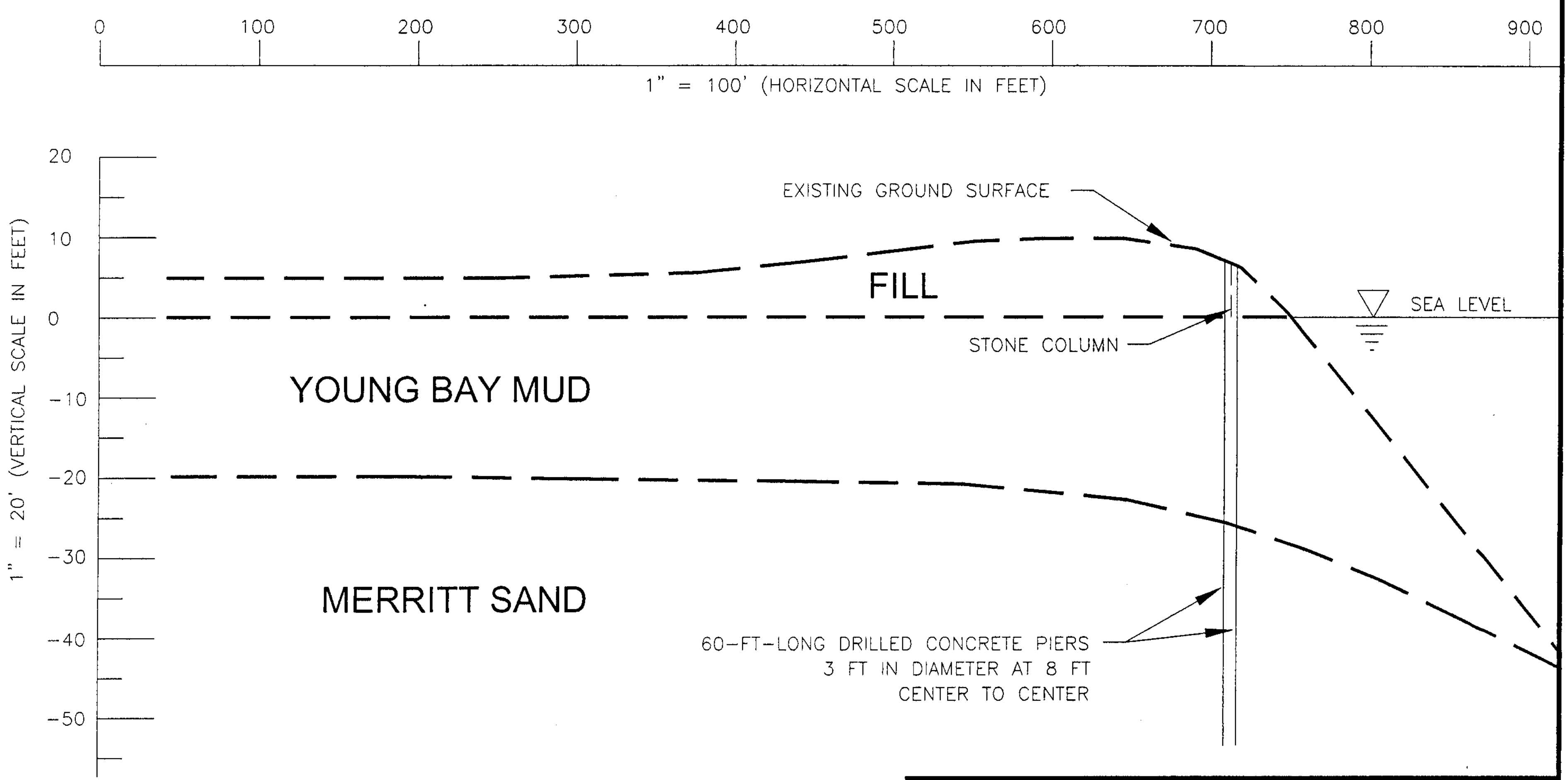
Figure 4-53
ALTERNATIVE 8 - DRILLED CONCRETE PIERS
WITH STONE COLUMNS (SECTION G-G')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-53.dwg 09/15/2003 10:49:36 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-54.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION H-H' (LOOKING WEST)

Figure 4-54
ALTERNATIVE 8 – DRILLED CONCRETE PIERS
WITH STONE COLUMNS (SECTION H-H')

Southwest Division
Naval Facilities Engineering Command

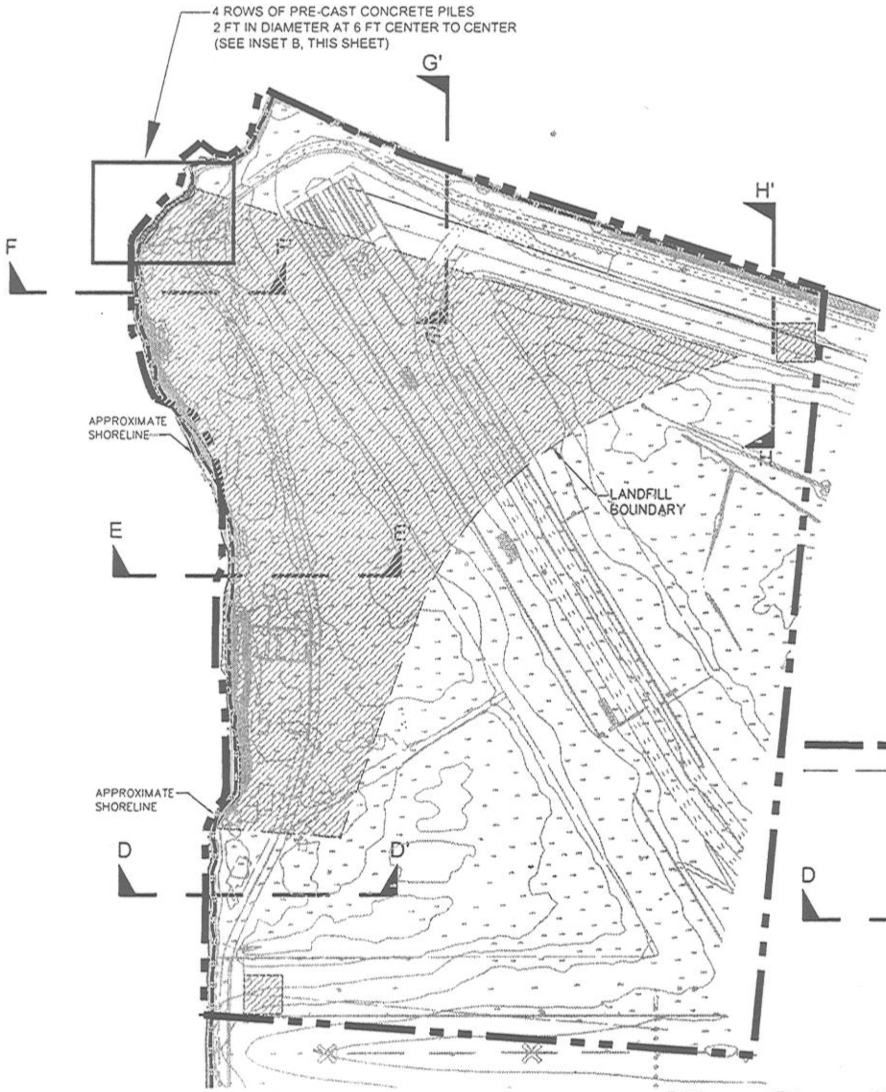
FOSTER WHEELER
ENVIRONMENTAL CORPORATION

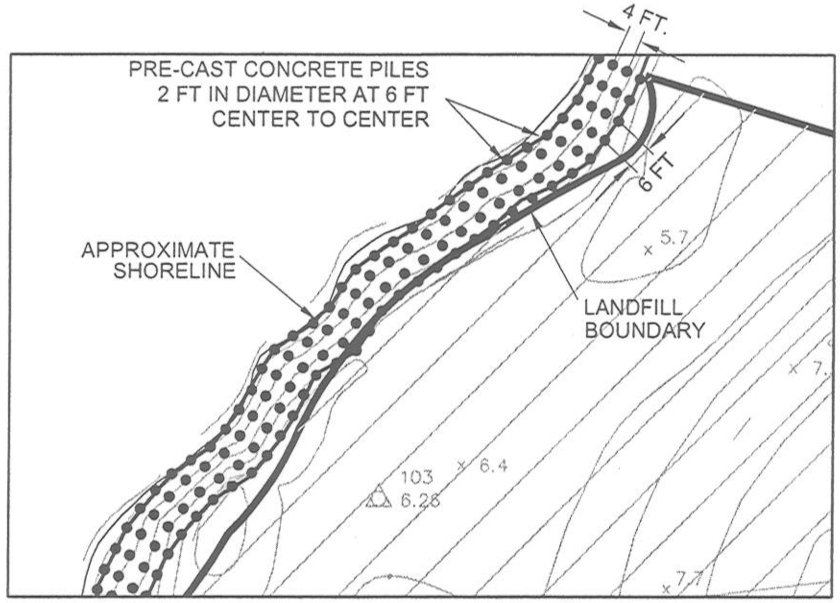
I:\1990-RAC\Cto-0054\Dwg\0313684-54.dwg 09/15/2003 10:52:21 AM PDT

I:\1990-RAC\CTO-0054\DWG\031368\0313684-55.DWG
PLOT/UPDATE: SEP 15 2003 10:53:03

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-55.DWG

REFERENCE:
HJW-GeoSpatial, Inc., Upland Topography
NAD27, NGVD29 - CCS Zone III





INSET - B

50 25 0 50
SCALE IN FEET

LEGEND

- SITE BOUNDARY
- LANDFILL BOUNDARY
- LANDFILL AREA
- DRILLED CONCRETE PIER
- D D' CROSS SECTION

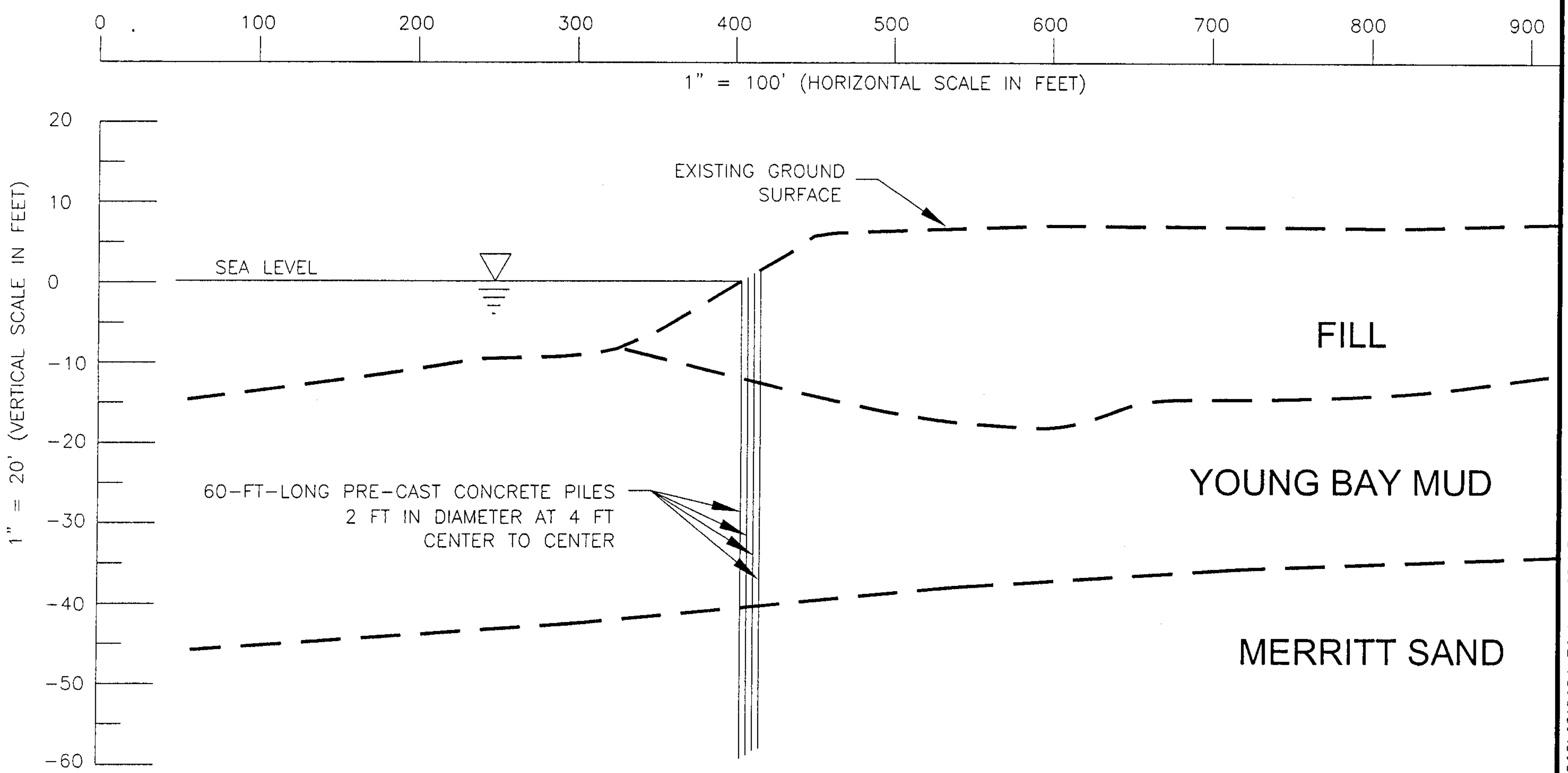
300 150 0 300
SCALE IN FEET

Figure 4-55
ALTERNATIVE 9: PRE-CAST CONCRETE PILES
(PLAN VIEW)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-56.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION D-D' (LOOKING NORTH)

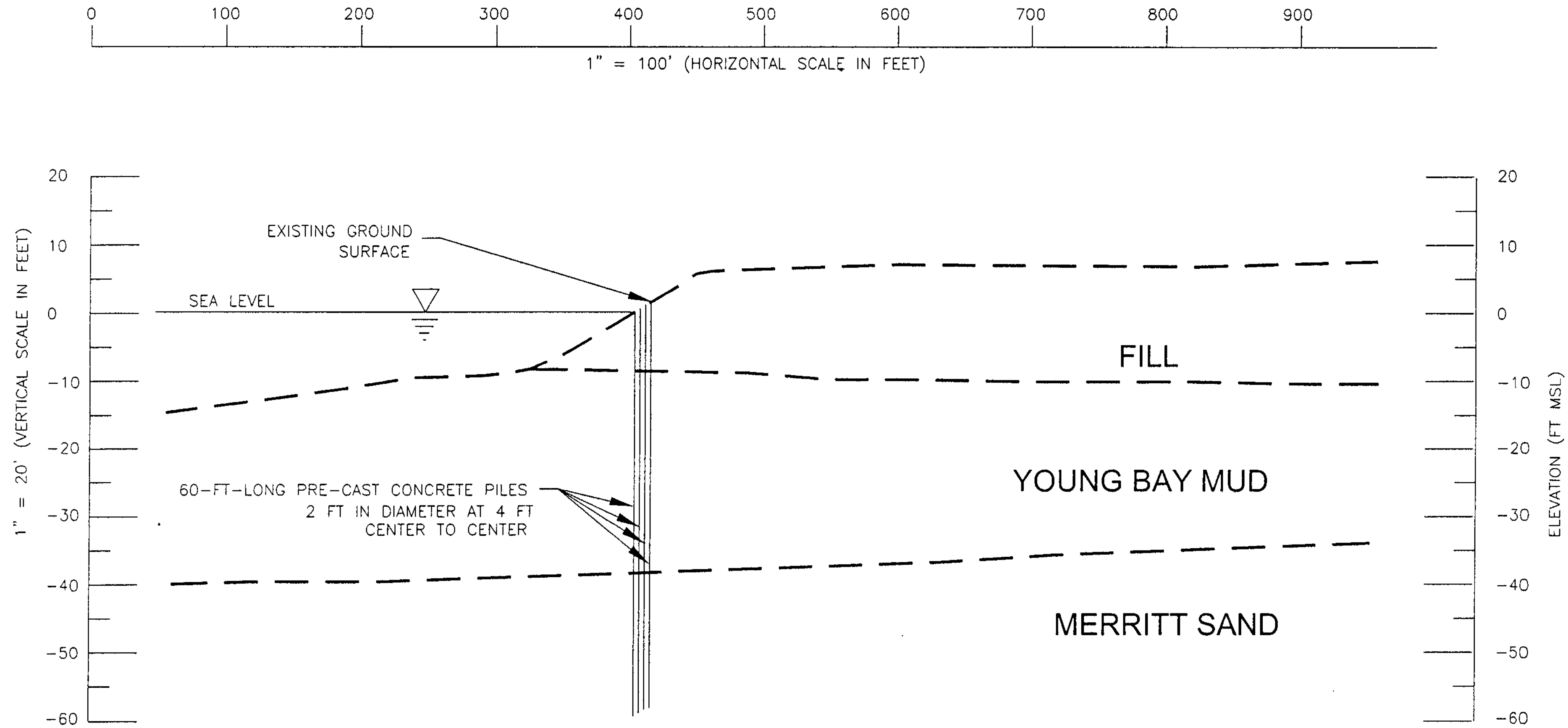
Figure 4-56
ALTERNATIVE 9 - PRE-CAST CONCRETE PILES WITH STONE COLUMNS (SECTION D-D')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-56.dwg 09/15/2003 10:56:08 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-57.DWG



SECTION E-E' (LOOKING NORTH)

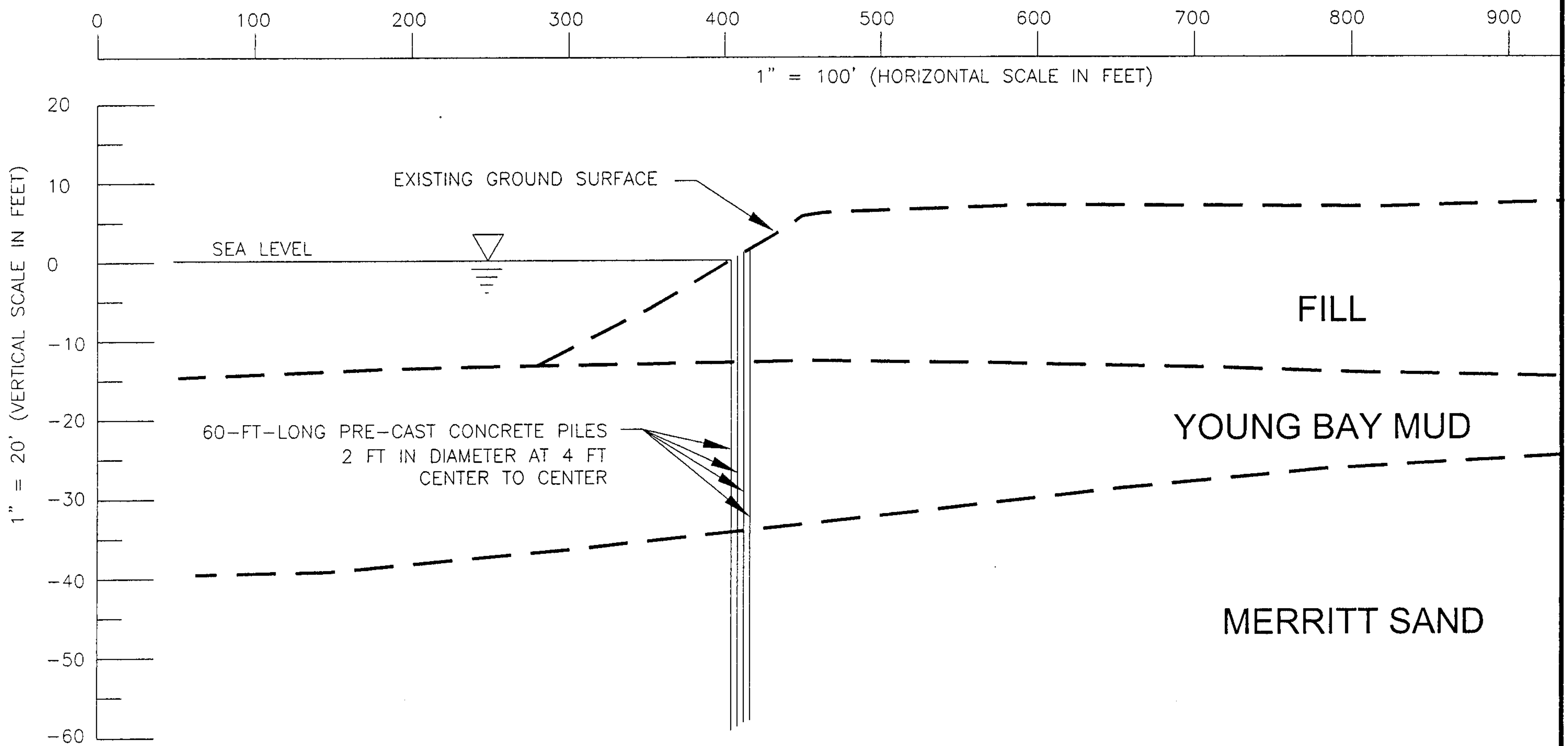
Figure 4-57
ALTERNATIVE 9 – PRE-CAST CONCRETE PILES (SECTION E-E')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-57.dwg 09/15/2003 10:57:38 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO: 0313684-58.DWG
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	



SECTION F-F' (LOOKING NORTH)

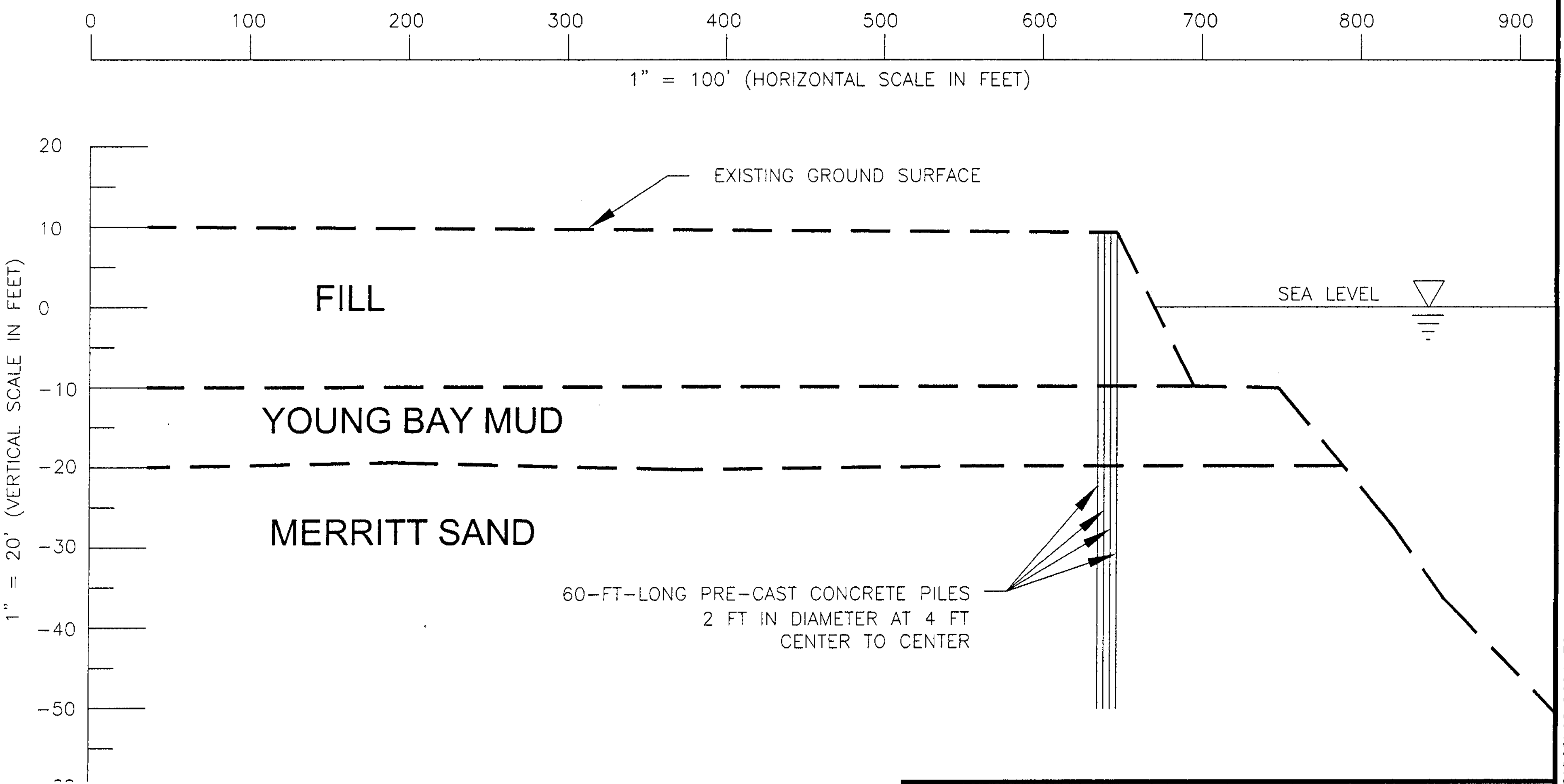
Figure 4-58
ALTERNATIVE 9 - PRE-CAST CONCRETE PILES
(SECTION F-F')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-58.dwg 09/15/2003 10:58:26 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-59.DWG



SECTION G-G' (LOOKING WEST)

Figure 4-59

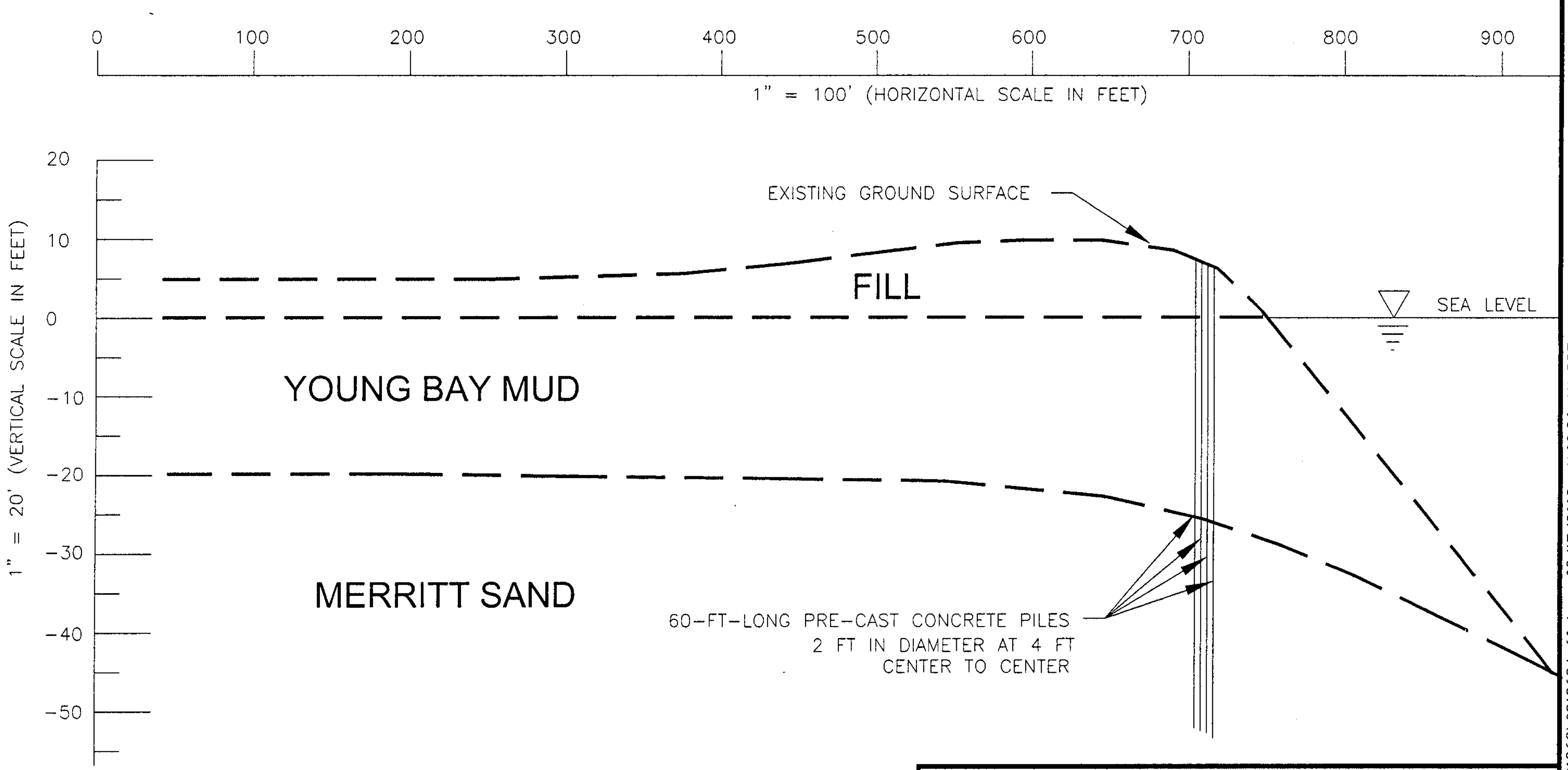
ALTERNATIVE 9 - PRE-CAST CONCRETE PILES
(SECTION G-G')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\0313684-59.dwg 09/15/2003 11:00:42 AM PDT

DRAWN BY: M.D.	CHECKED BY: T.L.	APPROVED BY: A.L.	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-60.DWG



SECTION H-H' (LOOKING WEST)

Figure 4-60
ALTERNATIVE 9 - PRE-CAST CONCRETE PILES
(SECTION H-H')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

I:\1990-RAC\Cto-0054\Dwg\031368\0313684-60.dwg 09/15/2003 11:02:36 AM PDT

permeated with bay water. Different types of cement (Type I, II, III, IV, V) would be used for the compatibility testing.

In order to ensure that the pre-cast concrete piles are to be driven plumb, a construction quality control program would be implemented.

4.2 INDIVIDUAL ANALYSIS OF ALTERNATIVES

This section provides a summary of individual analysis performed on the nine alternatives described in the previous section. The analysis involves a screening process based on implementability and cost evaluation criterion. Analysis methods used and results of the screening process are discussed below.

4.2.1 Implementability Analysis

Implementability analysis is based on the technical and administrative feasibility of the alternative. The administrative feasibilities of the selected nine alternatives were addressed in the initial screening process (see Table 3-3). Therefore, the main focus of implementability evaluation in this section is the technical feasibility of each alternative. The analyses performed for the implementability analysis included a static and seismic stability evaluation of each alternative. The alternatives that are determined to be technically feasible, or in other words, meet the performance criteria, are evaluated based on cost criteria in the next section.

The selected alternatives improve stability of the site perimeter slopes either by increasing shear strength of the site soils (particularly Young Bay Mud) and/or by providing a physical buttress.

The following subsections present 1) a discussion of the performance criteria application to each remedial alternative, 2) the analysis methods used to evaluate static and seismic stability of each alternative, and 3) the results of the stability analyses.

4.2.2 Performance Criteria Application

The performance criteria for evaluating the static and seismic stability of the site perimeter slopes were discussed in Section 2.3. This section summarizes the application of these criteria along with a discussion of the effects of seismically induced deformations on the structural integrity of the selected remedial alternatives.

As discussed in Section 2.3, the maximum allowable seismic displacement of the site perimeter slopes (the seismic stability performance criterion) was selected to be equal to 4 feet. This was based mainly on the width of the buffer zone between the limit of the waste and the shoreline. However, it should be noted that the characteristics of a remedial measure used to enhance stability of the perimeter slopes will also influence the selection of the seismic stability performance criterion. The seismically induced deformations of a remedial measure (a physical buttress) should not compromise its structural integrity. Therefore, the performance criterion for seismic stability should be based on the smaller value of the maximum allowable lateral displacements of the site perimeter berm and the remedial alternative.

The more massive remedial structures such as those that increase strength of the site soils by densification/consolidation or by addition and mixing of higher strength materials (for example, stone, cement, and so forth) are expected to tolerate relatively large deformations (as much as 5 to 8 feet) without losing their functionality. However, the slender remedial structures, such as sheet pile walls and drilled concrete piers acting as retaining structures, may not be able to withstand such deformations. Based on the above considerations, the following performance criteria for the screening/feasibility level evaluations were considered:

1. Static factor of safety for stability evaluation under long-term static loading conditions should be a minimum of 1.5.
2. Static factor of safety for temporary conditions (such as, during site pre-loading to consolidate Young Bay Mud) should be a minimum of 1.15.
3. Static factor of safety based on post-earthquake strength parameters should be greater than 1.0.
4. Maximum allowable seismically induced lateral displacement, which is a measure of seismic stability, should be less than 4 feet. More stringent performance criteria for seismic stability will be developed at the design stage for the selected alternative.

4.2.3 Analysis Methods

A detailed evaluation of geotechnical and seismic hazards at the site and the analysis methods used were presented in the RI Report Addendum, Volume III (FWENC, 2002). The following describes the analysis methods used to evaluate the technical feasibility of the nine selected remedial alternatives.

Global/Overall Stability

Static Stability Analyses. Conventional two-dimensional limit equilibrium stability analyses were performed to evaluate the global/overall stability of each of the nine alternatives. The computer program PC-STABL-5M (Achilleos, 1988) was used to calculate the factors of safety against potential failure. The program uses two-dimensional limit equilibrium theory to provide general solutions to slope stability problems. Both circular and non-circular potential sliding surfaces can be pre-specified or randomly generated. Modified Janbu method (Huang Y. H., 1983) and Modified Bishop method (Lambe and Whitman, 1969) of analysis were used for this study. Most critical surfaces identified during an initial extensive search based on the simplified Janbu method of analysis were subsequently analyzed using the more rigorous Spencer's method of analysis. The Modified Bishop and Janbu methods are considered less rigorous methods because they do not satisfy both force and moment equilibrium simultaneously. These methods are generally conservative compared with the more rigorous Spencer's method, and they typically result in lower factors of safety than the more rigorous methods (Duncan, 1992).

The most critical potential failure mechanism considered was either a circular failure or a wedge (block) failure plane starting at the landfill surface, passing through the proposed landfill cover and the existing underlying fill, and then sliding mostly within the Young Bay Mud toward San Francisco Bay or the Oakland Inner Harbor Channel and shearing through or below the remedial containment zone/structure provided along the shoreline to enhance stability, and finally ending on the bay floor surface.

For each remedial alternative, three different loading cases were analyzed for the selected analysis cross sections. These cases included: 1) static (long-term) stability analysis, 2) the post-earthquake static stability analysis, and 3) pseudo-static stability analysis to compute yield accelerations (the pseudo-static earthquake acceleration resulting in a factor of safety of approximately 1.0). The first case was analyzed using the long-term strength properties of the soil materials (see Table 1-1). The second (post-earthquake) case was analyzed using the residual shear strength properties of the Young Bay Mud and the liquefied granular soils [reduced strength properties due to strong ground shaking (see Table 1-1)], and the third case was analyzed using the long-term strength properties of the Young Bay Mud and the average value between long-term and post-earthquake properties for the sandy soils.

Analysis Sections. Five representative cross sections (Cross Sections D-D', E-E', and F-F' along the San Francisco Bay shoreline and Cross Sections G-G' and H-H' along the Oakland Inner Harbor) were selected to analyze stability of the site perimeter slopes (see Figure 4-1). Note that cross section labels were chosen arbitrarily and do not necessarily begin with A-A'. The results of static and seismic slope stability analyses demonstrated that the site perimeter slopes are not seismically stable, and in some areas, the factor of safety for static stability was calculated to be less than the minimum allowable value of 1.5 after installation of the proposed 4-foot-thick cover. Based on the results of the stability analyses (FWENC, 2002), Cross Sections D-D', F-F', and G-G' were selected as the most representative critical sections for the implementability analysis of the proposed remedial alternatives.

Potential Sliding Mass and Yield Acceleration Analyses. Yield accelerations (K_y) were subsequently computed from a series of pseudo-static analyses. Similar to the static cases, the pseudo-static slope stability analyses showed that the most critical potential failure mechanism considered is a circular failure or a wedge (block) failure plane sliding through the proposed landfill cover and the existing underlying fill, and then mostly through the Young Bay Mud layer and through or below the particular remedial containment zone/structure provided to enhance stability.

Seismically Induced Permanent Displacement Analyses

The effects of earthquake shaking on the site perimeter slopes prior to and after implementation of remedial alternatives were evaluated by estimating seismically induced permanent displacements using Newmark-type pseudo-dynamic double-integration deformation analysis methods (Newmark, 1965). Figure 4-61 [developed as part of the RI Report Addendum, Volume III (FWENC, 2002)] summarizes the results of the estimated seismically induced permanent displacement (δ) (computed using a Newmark-type double-integration method applied to the average acceleration time history of the potential sliding mass) versus the yield acceleration coefficient (K_y). Note that K_y is a fraction of the acceleration due to gravity, g (= 32.2 feet/second^2). These analyses use, as input, the average acceleration time history of the potential sliding mass estimated from the one-dimension dynamic SHAKE91 response analyses (Idriss and Sun, 1991).

The effect of the proposed improvements on ground motions computed using a one-dimension site response analysis method is considered to be minimal. Therefore, the correlation between seismically induced slope deformation and yield acceleration (shown in Figure 4-61) developed based on existing conditions, is still applicable.

A range of seismic deformations corresponding to yield acceleration coefficient values (K_y) between 0.11 and 0.16 is shown in Table 4-1.

TABLE 4-1

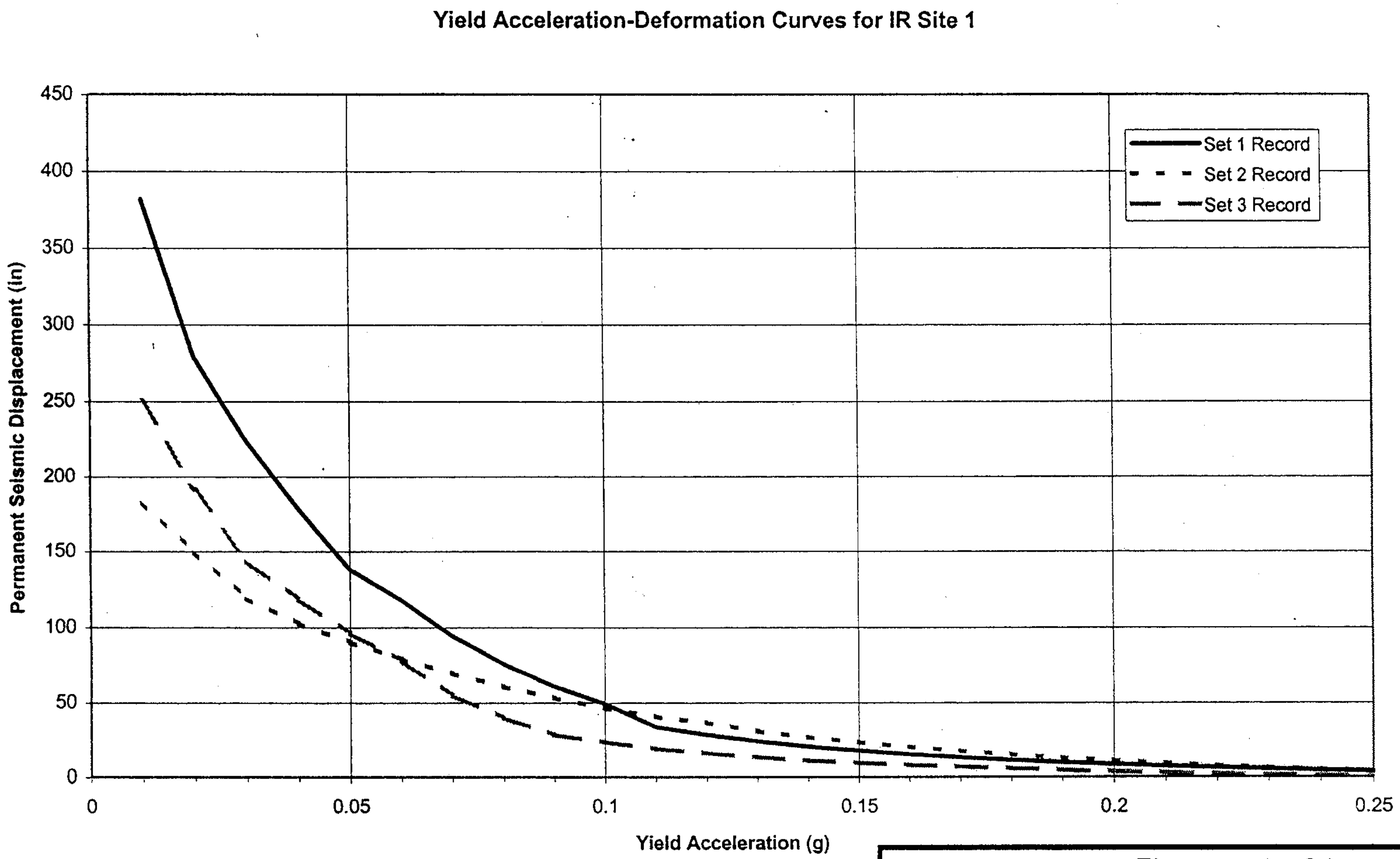
SEISMIC DEFORMATIONS CORRESPONDING TO YIELD ACCELERATION COEFFICIENT VALUES (K_y)

K_y	Seismic Deformation inches (feet)
0.11	47 (3.9)
0.12	41 (3.4)
0.13	36 (3.0)
0.14	31 (2.6)
0.15	26 (2.2)
0.16	23 (1.9)

This range indicates that yield acceleration coefficients less than 0.11 would result in seismic deformations greater than the performance criterion value of 4 feet, and yield acceleration coefficients equal or greater than 0.15 would result in seismic deformations less than .approximately 2 feet.

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-1368	DRAWING NO:
DATE: 09/26/03	REV: REVISION 0		CTO: # 0054	0313684-61.DWG

I:\1990-RAC\CTO-0054\DWG\031368\0313684-61.DWG
PLOT/UPDATE: SEP 15 2003 11:04:49



SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-61
SEISMICALLY INDUCED SLOPE DEFORMATIONS VERSUS YIELD ACCELERATION

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER ENVIRONMENTAL CORPORATION

Internal/Localized Stability

Following the global/overall stability evaluation for the proposed alternatives, internal/localized stability evaluations were performed for soils outboard of the improved soil zones (for example, the zone improved using stone columns and surcharge in Alternative 2), and for relatively slender retaining structures such as sheet piles, drilled concrete piers, and pre-cast concrete piles. Massive structures, such as stone columns or soil/cement walls, are internally stable.

Fill soils in front of the improved soil zone may be subject to flow slide instability due to the presence of the free slope. Slope stability analyses were performed to evaluate stability of the upper fill layer/improved soil zone if the fill outboard of the improved soil zone fails (for example, see Cross Sections D-D', F-F', and G-G' in Figures 4-10, 4-12, and 4-13, respectively). These analyses demonstrated that there may be a potential for shallow progressive post-earthquake instability along the north shore slopes. The local shallow or surficial instability/flow slide of the fill soils outboard of the improved soil zone can be addressed either by extending the selected remedial alternative offshore to include these fill soils, or by using riprap along the shoreline to enhance shallow stability of the fill soils outboard of the improved soil zone.

For sheet pile walls, the determination of the safe penetration length into the Merritt Sand, and deflections, shear forces, bending moments, and lateral loads as a function of depth were made using the ProSheet computer program developed by MegaTec Corporation (MegaTec Corporation, 1998). Calculations were performed to determine suitable sizes and grades of steel sheet piling for use as retaining structures acting as a cantilever and as an anchored wall. Initially, anchor forces were assumed to act horizontally at the top of the sheet pile to minimize excavation in the landfill area adjacent to the shoreline. Lateral load analyses are based on soil parameters included in Table 1-1.

The resistance to lateral loads on drilled concrete piers and pre-cast concrete piles was analyzed using the computer program LPILE developed by Ensoft, Inc. of Austin, Texas (Ensoft, Inc., 1999). The program computes deflection, shear forces, bending moment, and soil response as a function of depth in nonlinear soils. Soil behavior was modeled with p-y curves [representing the nonlinear relationship between lateral load (p) and deflection (y) for soil-pile system] internally generated by the computer program following published recommendations for loose sands (fill), soft clay (Young Bay Mud), and dense sand (Merritt Sand). The free pile-head boundary condition was used in the analyses.

Because the potential slope sliding surface would extend through the installed group of concrete piers or piles, pile lateral resistance will be developed. The pile lateral resistance depends on the lateral pile response due to the slope displacement away from the shoreline into San Francisco Bay. Incorporation of the pier/pile slope reinforcing effect in the analysis and determination of lateral pile response requires an iterative approach, as described below:

1. Define the geometry of the slope and assign strength parameters for the on-site soils.
2. Incorporate the slope reinforcing effects of the pier/pile group by computing an equivalent shear strength of the group by combining shear strength of the pile itself and the pile influence zone shear strength.
3. Perform slope stability analyses to determine the yield acceleration using the computer program PC-STABL-5M (Achilleos, 1988) and estimate the resulting slope displacement using the relationship shown in Figure 4-61.
4. Conduct lateral pile response analysis using the computer program LPILE PLUS, Version 3.0 (Ensoft, Inc., 1999), by imposing the calculated slope displacement obtained in Step 3 to determine the resulting pile lateral resistance at the sliding plane.
5. Repeat Steps 2 through 4 until convergence is achieved on the pile lateral resistance at the sliding plane.

4.2.4 Feasibility Analysis Results

The results of implementability analyses (slope stability, seismic displacements, and structural response calculations) for the nine alternatives analyzed are summarized in Tables 4-2 and 4-3. Detailed discussions of the analysis results for each alternative are provided in the following subsections. Slope stability input files and plots illustrating geometries of each alternative cross section, the potential failure surfaces evaluated, and the ten most critical potential failure planes searched by the program are presented in Appendix A. ProSheet output for sheet pile analysis and LPILE output for drilled concrete piers and pre-cast concrete piles analyses are also included in Appendix A.

Alternative 1: Wick Drains with Surcharge

Wick drains would be installed along a 95-foot-wide zone east of the shoreline as shown in Figures 4-1 through 4-7. Based on the SHANSEP (Stress History and Normalized Soil Engineering Properties) approach described by Ladd and Foott (1974), and a normalized static long-term undrained shear strength for normally consolidated condition $(S_u/\Phi_{v'})'_{NC}$ of 0.2 shown in Table 1-1, a surcharge of 18 feet high is required to increase the undrained shear strength of the Young Bay Mud from 500 pounds per square foot (present condition) to about 1,000 psf (consolidated strength near the shoreline and below the surcharge).

TABLE 4-2

SUMMARY OF SLOPE STABILITY ANALYSIS RESULTS

Alternative No.	Alternative Name	Analysis Section	Case	Static Factor of Safety (2)(3)(4)	Yield Acceleration K_y (g) and Seismic Permanent Displacement $\delta^{(5)}$ (feet)	
					K_y (1)	$\delta^{(5)}$
1	Wick Drains with Surcharge	D-D'	Static (long-term)	2.03[B]		
			Static (post-earthquake)	1.59[B]		
			Pseudo-static		0.11[J] 0.11[S]	3.9 3.9
			Static (18-foot-high surcharge)	1.02[B]		
2	Stone Columns with Surcharge	D-D'	Static (pre-loading)	1.14[B]		
			Static (long-term)	2.07[B]		
			Static (post-earthquake)	1.61[B]		
			Pseudo-static		0.12[B] 0.12	3.4 3.4 3.9
		F-F'	Static (long-term)	2.26[B]		
			Static (post-earthquake)	1.75[B]		
			Pseudo-static		0.12[J] 0.12[S]	3.4 3.4
		G-G'	Static (long-term)	1.88[B]		
			Static (post-earthquake)	1.76[B]		
			Pseudo-static		0.15[J] 0.15[S]	2.2 2.2

TABLE 4-2 (Continued)

SUMMARY OF SLOPE STABILITY ANALYSIS RESULTS

Alternative No.	Alternative Name	Analysis Section	Case	Static Factor of Safety (2)(3)(4)	Yield Acceleration K_y (g) and Seismic Permanent Displacement $\delta^{(5)}$ (feet)	
					K_y (1)	$\delta^{(5)}$
3	Sheet Piles with Anchors	D-D'	Static (long-term)	4.33[B]		
			Static (long-term)	4.54[S]		
			Static (post-earthquake)	4.06[B]		
			Static (post-earthquake)	4.13[S]		
			Pseudo-static		0.31[B] 0.27[S]	0.2 0.2
4	Stone Columns with Surcharge and Sheet Piles	D-D'	Static (long-term)	4.34[B]		
			Static (long-term)	4.39[S]		
			Static (post-earthquake)	4.08[B]		
			Static (post-earthquake)	4.14[S]		
			Pseudo-static		0.31[B] 0.29[S]	0.2 0.2

TABLE 4-2 (Continued)

SUMMARY OF SLOPE STABILITY ANALYSIS RESULTS

Alternative No.	Alternative Name	Analysis Section	Case	Static Factor of Safety (2)(3)(4)	Yield Acceleration K_y (g) and Seismic Permanent Displacement $\delta^{(5)}$ (feet)	
					K_y (1)	$\delta^{(5)}$
5	Soil Cement Gravity Wall and Stone Columns	D-D'	Static (long-term)	3.03[B]		
			Static (long-term)	3.05 [S]		
			Static (post-earthquake)	2.13[B]		
			Static (post-earthquake)	2.36[S]		
			Pseudo-static (front) Pseudo-static Pseudo-static Pseudo-static		0.12[B] 0.16[B] 0.15[J] 0.15[S]	3.4 1.9 2.2 2.2
		F-F'	Static (long-term)	2.73[B]		
			Static (post-earthquake) (front)	2.31[B]		
			Static (post-earthquake)	2.37[S]		
			Pseudo-static Pseudo-static		0.15[J] 0.15[S]	2.2 2.2
		G-G'	Static (long-term)	1.90[B]		
			Static (post-earthquake)	2.12[S]		
			Static (post-earthquake)	1.69[B]		
			Static (post-earthquake)	1.75[S]		
			Pseudo-static Pseudo-static Pseudo-static Pseudo-static		0.22[J] 0.21[S] 0.18[B] 0.19[S]	.0.8 0.9 1.7 1.4

TABLE 4-2 (Continued)

SUMMARY OF SLOPE STABILITY ANALYSIS RESULTS

Alternative No.	Alternative Name	Analysis Section	Case	Static Factor of Safety (2)(3)(4)	Yield Acceleration K_y (g) and Seismic Permanent Displacement $\delta^{(5)}$ (feet)	
					K_y (1)	$\delta^{(5)}$
6	Concrete Wall	D-D'	Static (long-term)	3.50[B]		
			Static (long-term)	3.56[S]		
			Static (post-earthquake)	3.17[B]		
			Static (post-earthquake)	3.24[S]		
			Pseudo-static Pseudo-static		0.25[B] 0.23[S]	0.5 0.6
7	Excavation with Riprap	D-D'	Pseudo-static, riprap bottom above Merritt Sand		0.03[B]	17
			Static (long-term)	2.55[J]		
			Static (long-term)	2.71[B]		
			Static (post-earthquake)	2.07[J]		
			Static (post-earthquake)	2.19[B]		
			Pseudo-static Pseudo-static Pseudo-static		0.12[J] 0.12[S] 0.13[B]	3.4 3.4 3.0
8	Drilled Concrete Piers with Stone Columns	D-D'	Static (long-term)	4.34[B]		
			Static (long-term)	4.39[S]		
			Static (post-earthquake)	4.06[B]		
			Static (post-earthquake)	4.13[S]		
			Pseudo-static		0.14[S]	2.6

TABLE 4-2 (Continued)

SUMMARY OF SLOPE STABILITY ANALYSIS RESULTS

Alternative No.	Alternative Name	Analysis Section	Case	Static Factor of Safety (2)(3)(4)	Yield Acceleration K_y (g) and Seismic Permanent Displacement $\delta^{(5)}$ (feet)	
					K_y (1)	$\delta^{(5)}$
9	Pre-cast Concrete Piles	D-D'	Static (long-term)	4.34[B]		
			Static (long-term)	4.39[S]		
			Static (post-earthquake)	4.06[B]		
			Static (post-earthquake)	4.13[S]		
			Pseudo-static		0.16[S]	1.9

Notes:

(1) K_y Yield acceleration, defined as the value of the horizontal acceleration resulting in a pseudo-static factor of safety equal to unity

(2) [S] Spencer's "rigorous" method of analysis, used for most critical cases and loading conditions

(3) [J] Modified Janbu method of analysis, used for preliminary extensive searches for potential critical slip surfaces (Rankine blocks/wedges) and for cases where Spencer's method of analysis did not converge

(4) [B] Modified Bishop method of analysis, used for extensive searches for potential critical slip surfaces (circular)

(5) δ Seismically induced permanent displacement computed based on the procedure using the Newmark double-integration method of analysis (Newmark, 1965)

(g) acceleration due to gravity

TABLE 4-3

SUMMARY OF SHEET PILE WALL ANALYSIS RESULTS

Analysis Cross Section	Case	Remarks	Maximum Deflection (inches)	Anchor Force (kips per linear foot)
[D-D']	Static	60-foot cantilevered wall, post-earthquake soil properties	12.0	N/A
	Static	60-foot anchored wall, post-earthquake soil properties	0.6	5.5
	Static	60-foot cantilevered wall backed by 20-foot-wide stone column zone, post-earthquake soil properties	2.6	N/A
	Static	60-foot anchored wall backed by 20-foot-wide stone column zone, post-earthquake soil properties	0.1	3.1
	Seismic	60-foot cantilevered wall backed by 20-foot-wide stone column zone, averaged long-term and post-earthquake soil properties, 10 H psf seismic load	27.0	N/A
	Seismic	60-foot anchored wall backed by 20-foot-wide stone column zone, averaged long-term and post-earthquake soil properties, 20 H psf seismic load	1.9	22.0
	Seismic	60-foot wall w/ 5 kip/foot anchor backed by 20-foot-wide stone column zone, averaged long-term and post-earthquake soil properties, 10 H psf seismic load	10.5	5.0
[F-F']	Static	50-foot cantilevered wall, post-earthquake soil properties	5.5	N/A
	Static	50-foot anchored wall, post-earthquake soil properties	0.3	4.5
	Static	50-foot cantilevered wall backed by 20-foot-wide stone column zone, post-earthquake soil properties	0.3	N/A
	Static	50-foot anchored wall backed by 20-foot-wide stone column zone, post-earthquake soil properties	0.1	0.9
	Seismic	50-foot cantilevered wall backed by 20-foot-wide stone column zone, averaged long-term and post-earthquake soil properties, 10 H psf seismic load	10.5	N/A
	Seismic	50-foot anchored wall backed by 20-foot-wide stone column zone, averaged long-term and post-earthquake soil properties, 20 H psf seismic load	0.8	15.1

TABLE 4-3 (Continued)

SUMMARY OF SHEET PILE WALL ANALYSIS RESULTS

Analysis Cross Section	Case	Remarks	Maximum Deflection (inches)	Anchor Force (kips per linear foot)
[G-G']	Static	45-foot cantilevered wall, post-earthquake soil properties	5.6	N/A
	Static	45-foot anchored wall, post-earthquake soil properties	0.3	4.7
	Static	45-foot cantilevered wall backed by 20-foot-wide stone column zone, post-earthquake soil properties	< 0.1	N/A
	Static	45-foot anchored wall backed by 20-foot-wide stone column zone, post-earthquake soil properties	< 0.1	0.1
	Seismic	45-foot cantilevered wall backed by 20-foot-wide stone column zone, averaged long-term and post-earthquake soil properties, 10 H psf seismic load	10.6	N/A
	Seismic	45-foot anchored wall backed by 20-foot-wide stone column zone, averaged long-term and post-earthquake soil properties, 20 H psf seismic load	0.9	14.3

Notes:

psf – pounds per square foot
H – height of sheet pile
kip/foot – kips per linear foot
N/A – not applicable

Global/Overall (Slope) Stability Analysis. Global/overall slope stability analysis results for Sections D-D' are shown in Figure 4-62 and summarized in Table 4-2. As shown in Table 4-2 and Figure 4-61, a yield acceleration value of 0.11g would result in seismic deformations of less than the allowable performance limit of 4 feet. The long-term and post-earthquake static factors of safety are greater than the performance criteria of 1.5 and 1.0, respectively. The computed static factor of safety during the later stages of application of the required 18-foot-high surcharge (Section D-D') is 1.02. The minimum value based on the design criteria for temporary conditions is 1.15. Because of these considerations, this alternative is considered unstable (during pre-loading) and technically not feasible. Further analysis is not warranted. The analysis results are included in Appendix A1.

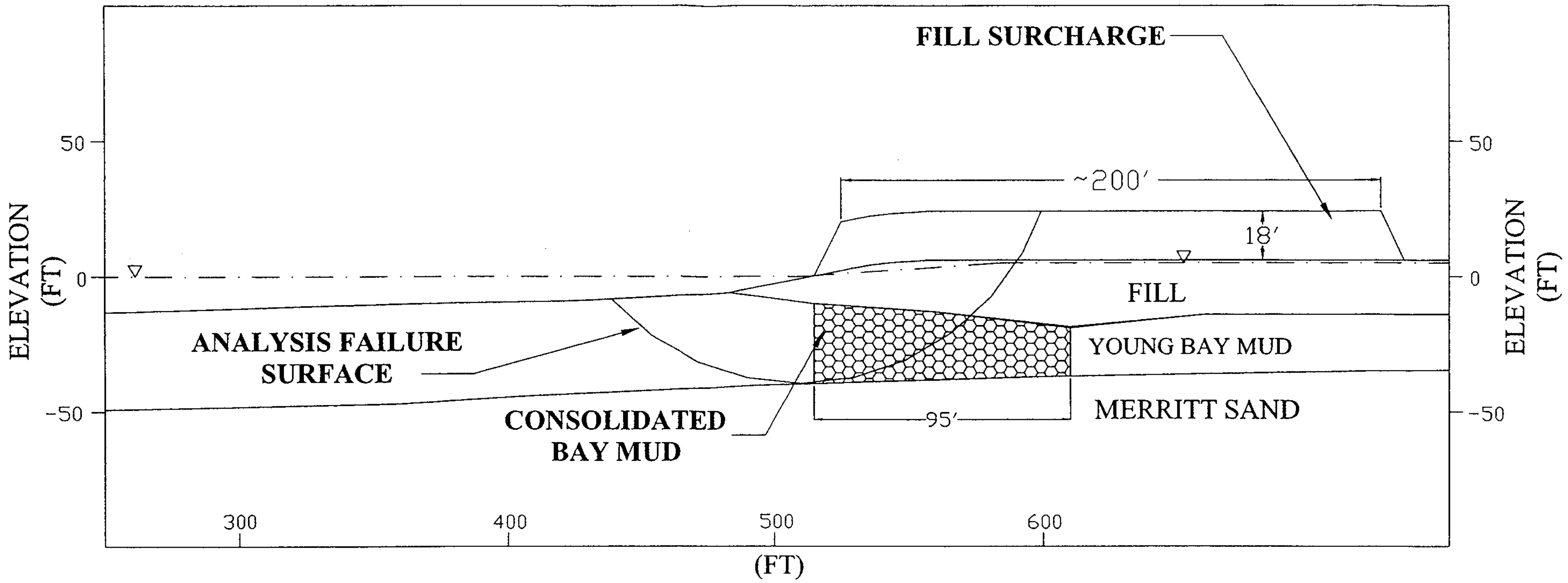
Alternative 2: Stone Columns with Surcharge

Stone columns would be installed along the shoreline as shown in Figure 4-8. A typical arrangement is shown in Figure 4-9. The diameter of individual columns and typical spacing along the shoreline used in the analysis are 3 and 8 feet, respectively. Cross sections are shown in Figures 4-10 through 4-14.

Global/Overall (Slope) Stability Analysis. Global/overall slope stability analysis results for Sections D-D', F-F', and G-G' are shown in Figures 4-63 and 4-64 and summarized in Table 4-2. Figures 4-63 and 4-64 show all the elements modeled in the slope stability analysis, whereas Figures 4-10 through 4-14 depict general conditions. Subsurface conditions are the same on both sets of figures. For analysis, the stone material was assumed to have an angle of internal friction of 40 degrees. The value of a 40-degree angle of internal friction is typical of dense gravel. This value is presented in the literature as a typical value for the design of stone columns. A local contractor also confirmed that this value is a good approximation of the anticipated field conditions based on their experience with similar applications. As shown in Figure 4-61, a yield acceleration value of 0.11g (see Figure 4-63) results in 3.9 feet of seismic deformation, which is smaller than the maximum allowable deformation of 4 feet established in Section 4.2.2. A stone column zone wall wider than the proposed analysis value of 38 feet would not improve stability significantly, and six rows of stone columns used in another analysis appear to be excessive. Post-earthquake static factors of safety are adequate for all of the cross sections analyzed (Figures 4-63 and 4-64). Based on the stability results, this alternative is technically feasible, and further analysis is warranted. Analysis results are included in Appendix A2.

Alternative 3: Sheet Piles with Anchors

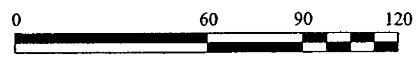
Sheet piles would be installed along the shoreline as shown in Figure 4-15. A typical sheet pile arrangement, showing the anchors required to restrict movement at the top of the sheet pile, is shown in Figure 4-16.



SECTION D-D'

WICK DRAIN

Bishop Method, Last Stage of Applying the Surcharge, Static **F.S. = 1.02**



SCALE
1" = 60'

DRAWING NO: 0313684-62.DWG	
DCN: FWSD-RAC-03-1368	CTO: # 0054
APPROVED BY: AL	
CHECKED BY: TL	REV: REVISION 0
DRAWN BY: MD	DATE: 09/26/03

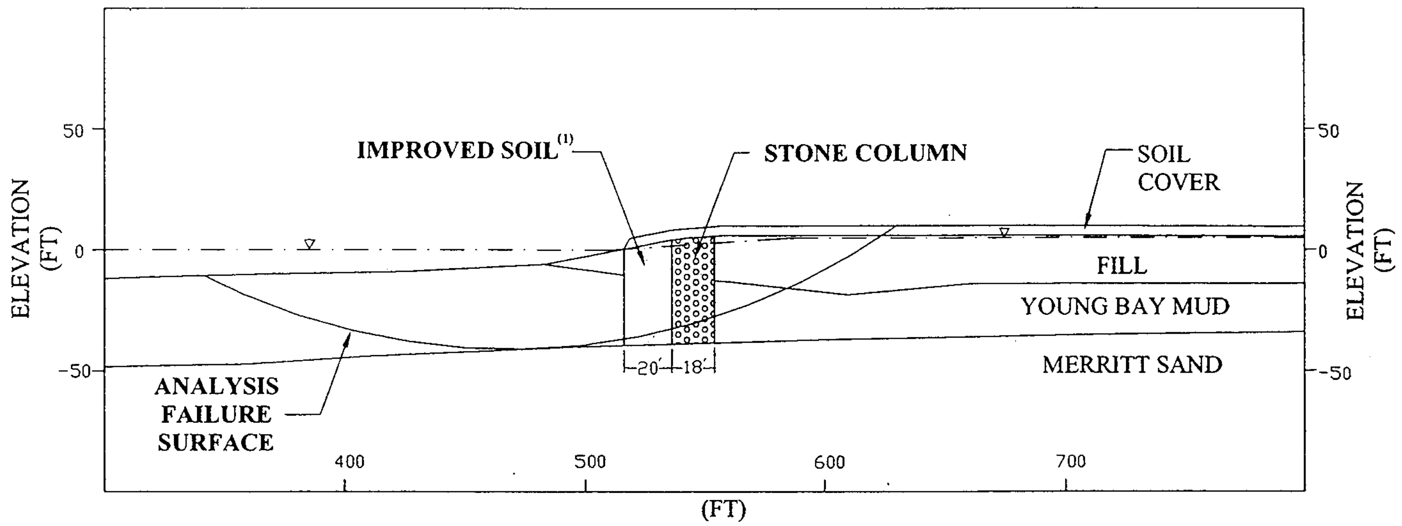
I:\1990-RAC\CTO-0054\DWG\031368\0313684-62.DWG
PLOT/UPDATE: SEP 15 2003 11:06:47

Figure 4-62

STABILITY ANALYSIS RESULTS FOR ALTERNATIVE 1 DURING PRELOADING (WICK DRAINS WITH SURCHARGE)

Southwest Division
Naval Facilities Engineering Command

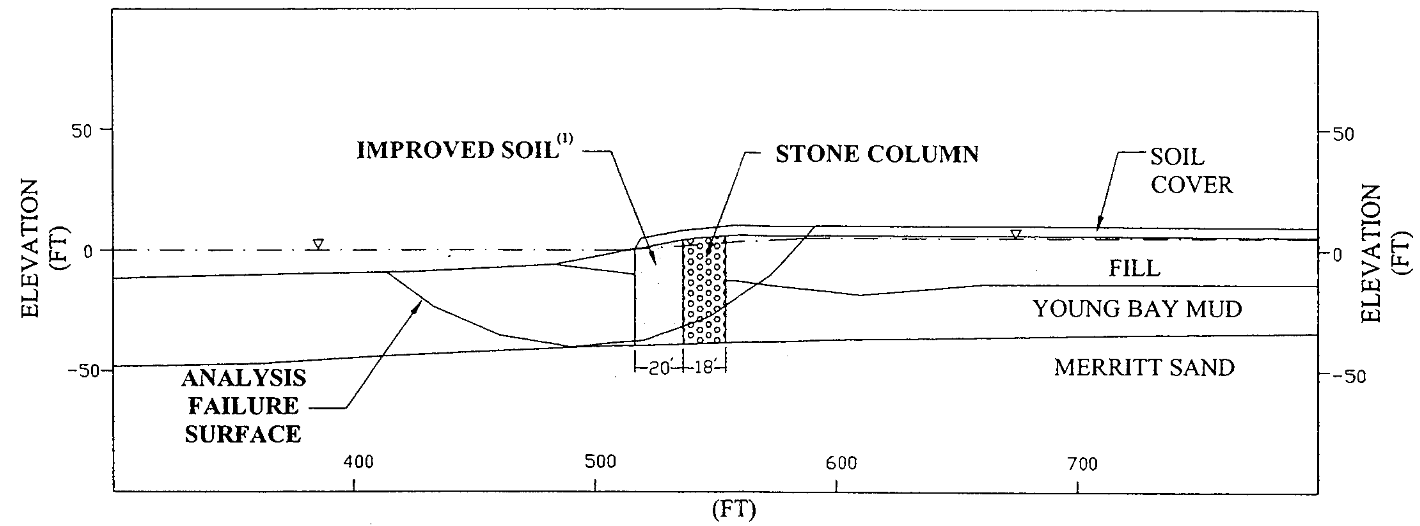
FOSTER WHEELER ENVIRONMENTAL CORPORATION



SECTION D-D'

STONE COLUMN

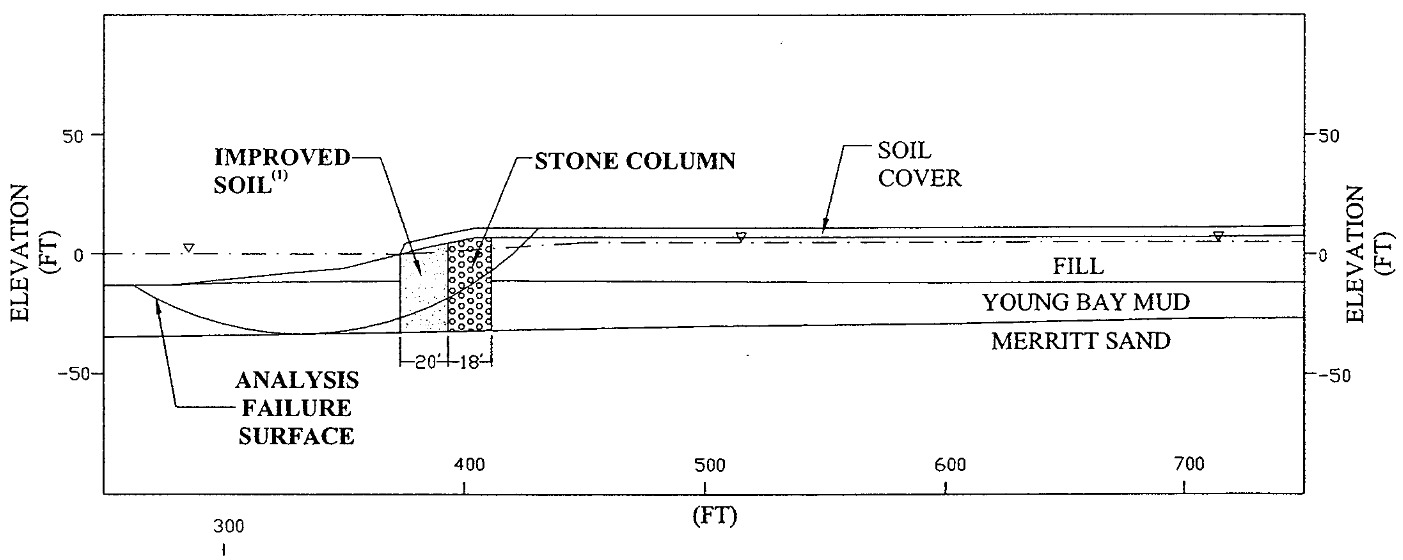
Spencer Method, Dynamic, K_y = **0.11 g** (2)



SECTION D-D'

STONE COLUMN

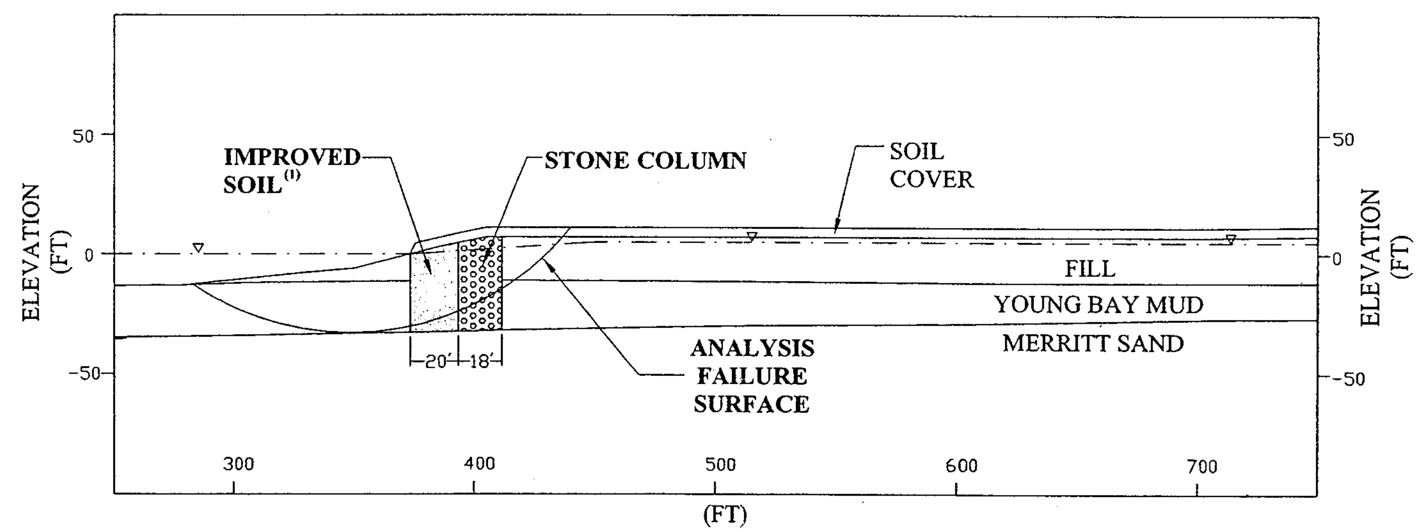
Bishop Method, Post Earthquake, Static **F.S. = 1.61** (2)



SECTION F-F'

STONE COLUMN

Spencer Method, Dynamic, K_y= **0.12 g** (2)



SECTION F-F'

STONE COLUMN

Bishop Method, Post-Earthquake, Static **F.S. = 1.75** (2)

Notes:

(1) Improved soil zone models the densified/improved soil matrix within the stone column area.

(2) The above stability analyses present the site stability evaluation following the implementation of remedial alternatives and include the effect of a future 4-foot soil cover.

0 80' 120' 160'
SCALE
1" = 80'

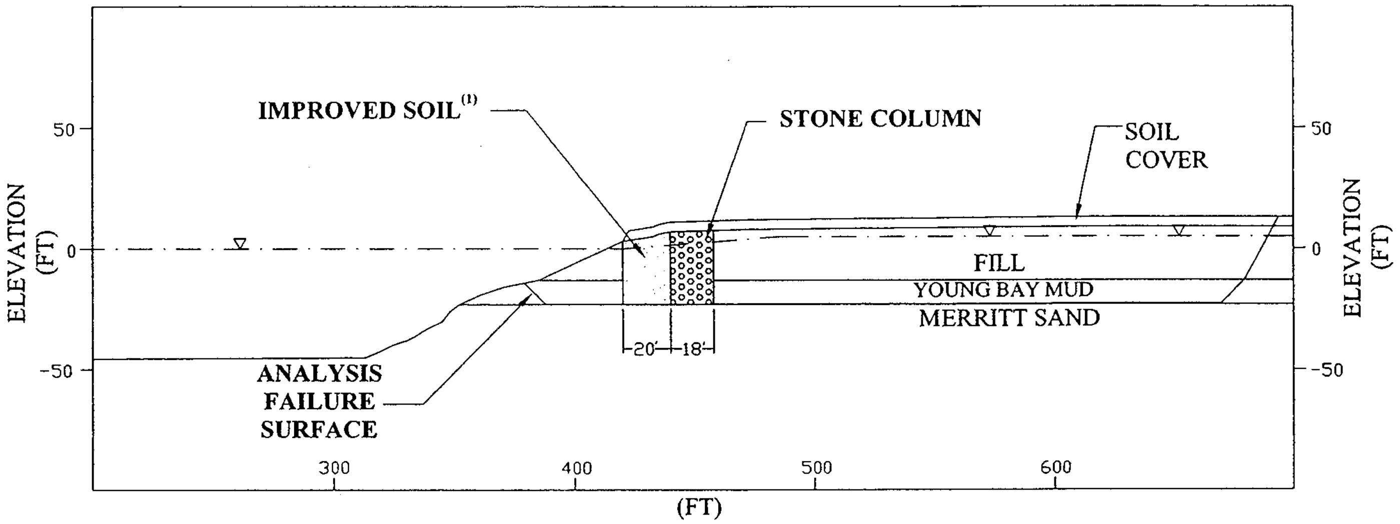
Figure 4-63
STABILITY ANALYSIS RESULTS FOR ALTERNATIVE 2 (STONE COLUMNS WITH SURCHARGE – SECTIONS D-D" AND F-F')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWING NO: 0313684-63.DWG
DCN: FWSD-RAC-03-1368
CTO: # 0054
APPROVED BY: AL
CHECKED BY: TL
REV: REVISION 0
DRAWN BY: MD
DATE: 09/26/03

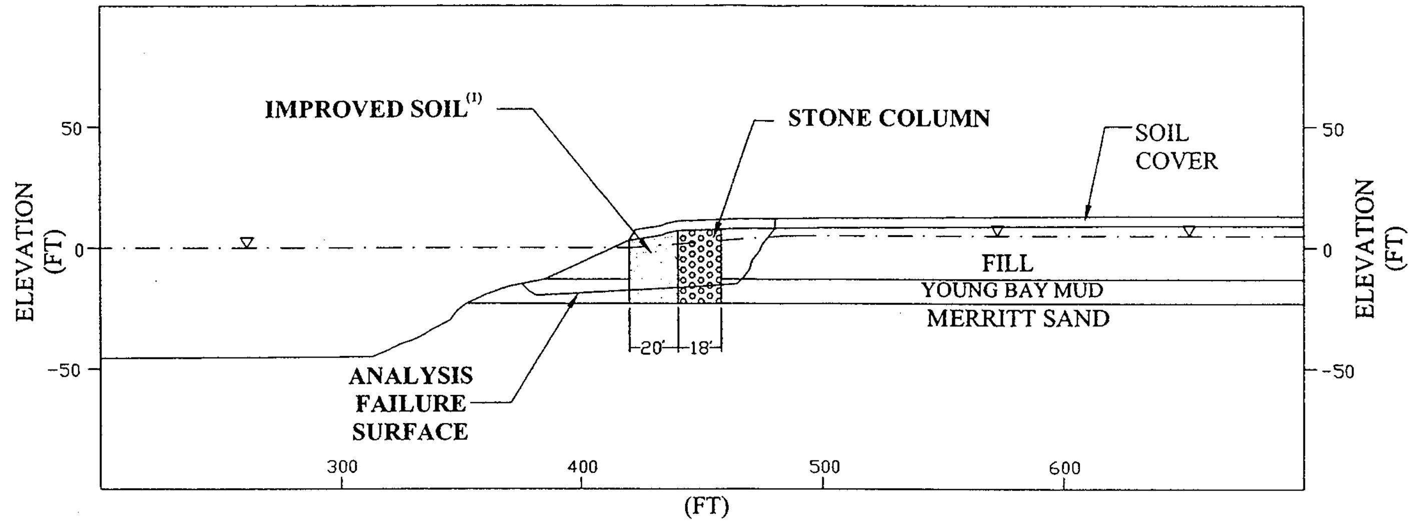
I:\1990-RAC\CTO-0054\DWG\031368\0313684-63.DWG
PLOT/UPDATE: SEP 15 2003 11:08:23



SECTION G-G'

STONE COLUMN

Spencer Method, Dynamic, K_y = **0.15 g** [2]



SECTION G-G'

STONE COLUMN

Bishop Method, Post-Earthquake, Static **F.S. = 1.76** [2]

0 80' 120' 160'

SCALE
1" = 80'

Notes:

(1) Improved soil zone models the densified/improved soil matrix within the stone column area.

(2) The above stability analyses present the site stability evaluation following the implementation of remedial alternatives and include the effect of a future 4-foot soil cover.

Figure 4-64
STABILITY ANALYSIS RESULTS FOR ALTERNATIVE 2
(STONE COLUMNS WITH SURCHARGE – SECTIONS G-G')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWING NO: 0313684-64.DWG
DCN: FWSD-RAC-03-1368
CTO: # 0054
APPROVED BY: AL
CHECKED BY: TL
REV: REVISION 0
DRAWN BY: MD
DATE: 09/26/03

I:\1990-RAC\CTO-0054\DWG\031368\0313684-64.DWG
PLOT/UPDATE: SEP 15 2003 11:09:58

Global/Overall (Slope) Stability Analysis. Global/overall slope stability analysis results are summarized in Table 4-2. A yield acceleration of 0.27g results in minimal seismic deformations along the potential failure surface (Figure 4-65). Computed static factors of safety for long-term and post-earthquake conditions are greater than 1.5 and 1.0, respectively.

Internal/Localized Stability Analysis. Analysis profiles for all cases analyzed (cantilevered and anchored conditions for Sections D-D', F-F', and G-G') are shown in Figures 4-17 through 4-21. Analysis results are summarized in Table 4-3. Detailed analysis sections and ProSheet computer program output are presented in Appendix A3. A maximum deflection of 12 inches for the cantilevered/post-earthquake static loading condition (Section D-D') at the top of the sheet pile is considered excessive for a permanent structure; therefore, a cantilevered sheet pile is not adequate. If the sheet pile is anchored at the top to restrict lateral deflections, an anchor force of 5.5 kips per linear foot of wall is developed at the ground surface under a static (post-earthquake) condition. This force is considered excessive and more than one line of anchors, buried into the disposal area, may be required.

This alternative is technically feasible and further analysis is warranted if the corrosion and anchor force issues are addressed in an environmentally safe and cost-effective manner. The analysis results are included in Appendix A3.

Alternative 4: Stone Columns with Surcharge and Sheet Piles

Stone columns and sheet piles would be installed along the shoreline as shown in Figure 4-22. The stone column zone behind the sheet piles is approximately 20 feet wide. A typical configuration is shown in plan view in Figure 4-22.

Global/Overall (Slope) Stability Analysis. The analysis results for Alternative 3 are applicable for this case since both alternatives are similar. Based on Alternative 3 results, yield acceleration and static factors of safety values are adequate, and further analysis is not required.

Internal/Localized Stability Analysis. Analysis profiles for all cases analyzed (cantilevered and anchored conditions for Sections D-D', F-F', and G-G') are shown in Figures 4-24 through 4-28. Analysis results are summarized in Table 4-3. Detailed analysis sections and ProSheet computer program output are presented in Appendix A4. A maximum deflection for cantilevered/post-earthquake static loading of approximately 3 inches (Section D-D') at the top of the sheet pile is considered adequate for permanent structures. As indicated in Table 4-3, if the sheet pile is anchored at the top to restrict lateral deflections, an anchor force of 3.1 kips per linear foot of wall is developed at the ground surface. The magnitude of this force is considered to be very high. For the case of seismic loading condition, 10.5 inches of deflection were estimated, assuming anchors are installed at the ground surface with an anchor force of approximately 5.0 kips per linear foot. Because of this large force, two lines of anchors may be required to establish anchor spacing in the range of 5 to 10 feet. Also, to develop resistance to these high anchor forces, the anchors may need to be inclined about 45 degrees from the horizontal plane into the landfill materials, which involves excavation within the landfill area.

This alternative is technically feasible and further analysis is warranted if the corrosion and anchor force issues are addressed in an environmentally safe and cost-effective manner. The analysis results are included in Appendix A4.

Alternative 5: Soil Cement Gravity Wall and Stone Columns

The soil cement gravity wall would be approximately 24 feet wide and installed a minimum of 5 feet into the Merritt Sand along the shoreline, as shown in Figure 4-29 and 4-30. This alternative considers mixing the Young Bay Mud material with a slurry of cement using large-diameter augers to inject and mix the cement as described in Section 4.1.5. Unconfined compression strength values of the soil cement mix would range from 50 to 100 psi. Based on these values, assumed shear strength for the mix is 5.0 kips per square feet. The fill overlying the Young Bay Mud would be improved with stone columns. Typical profiles (Sections D-D', E-E', F-F', G-G', and H-H') were shown in Figures 4-31 through 4-35.

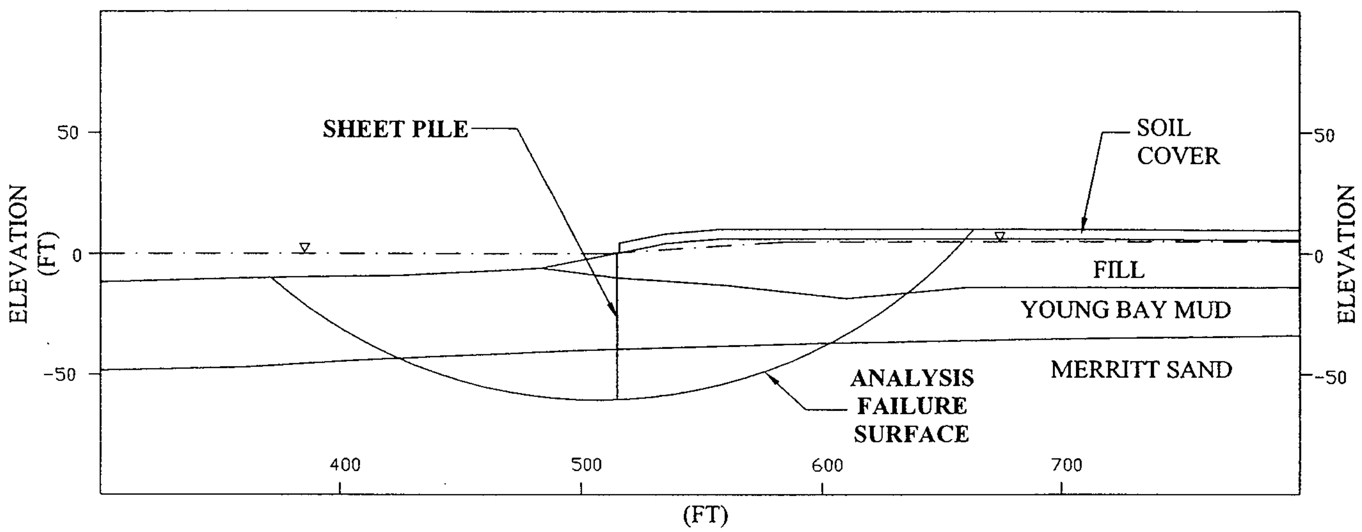
Global/Overall (Slope) Stability Analysis. Global/overall slope stability analysis results for the critical Sections D-D', F-F', and G-G' are presented in Figures 4-66 and 4-67 and summarized in Table 4-2. A computed minimum yield acceleration of 0.15g results in 2.2 feet of seismic deformations along the potential failure surface (see Figure 4-61), which is below the performance criteria of 4 feet. Computed static factors of safety for long-term and post-earthquake conditions are greater than 1.5 and 1.0, respectively.

Internal/Localized Stability Analysis. The soil cement gravity wall is a massive structure subjected to relatively similar lateral pressures on both sides of the wall, which makes this configuration stable under both static and seismic loading conditions. Because of the relatively small slenderness ratio, this wall is not subjected to flexion.

This alternative is technically feasible, and further analysis is warranted. The analysis results are included in Appendix A5.

Alternative 6: Concrete Wall

The concrete gravity wall would be approximately 14 feet wide and installed a minimum of about 5 feet into the Merritt Sand along the shoreline, as shown in Figure 4-36 and 4-37. This alternative consists of excavating a trench and backfilling the trench with concrete. It is anticipated that the unconfined compressive strength of concrete would range between 500 and 1,000 psi. A shear strength value of 36 kips per square foot, equal to half of the minimum unconfined compressive strength, was used in the analysis. Typical profiles (Sections D-D', E-E', F-F', G-G', and H-H') were shown in Figures 4-38 through 4-42.

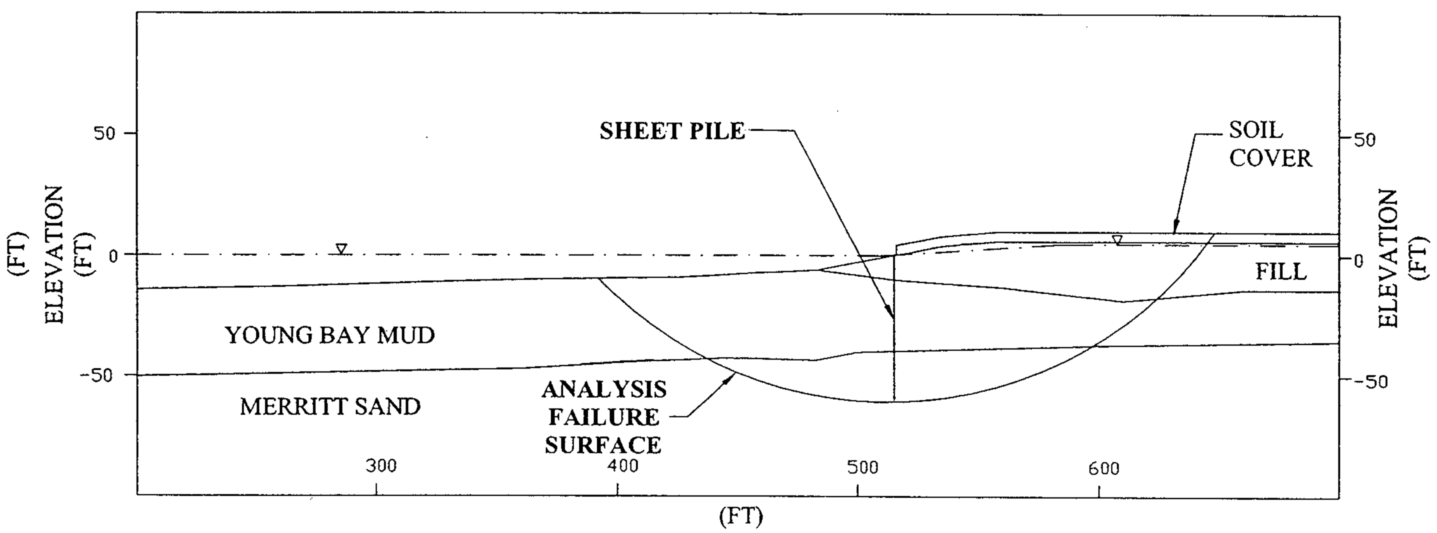


SECTION D-D'

SHEET PILE

Spencer Method, Dynamic, **K_y= 0.27 g**[(1)]

0 80' 120' 160'
SCALE
1" = 80'



SECTION D-D'

SHEET PILE

Bishop Method, Post Earthquake, Static **F.S. = 4.13**[(1)]

Notes:

[(1)] The above stability analyses present the site stability evaluation following the implementation of remedial alternatives and include the effect of a future 4-foot soil cover.

Figure 4-65

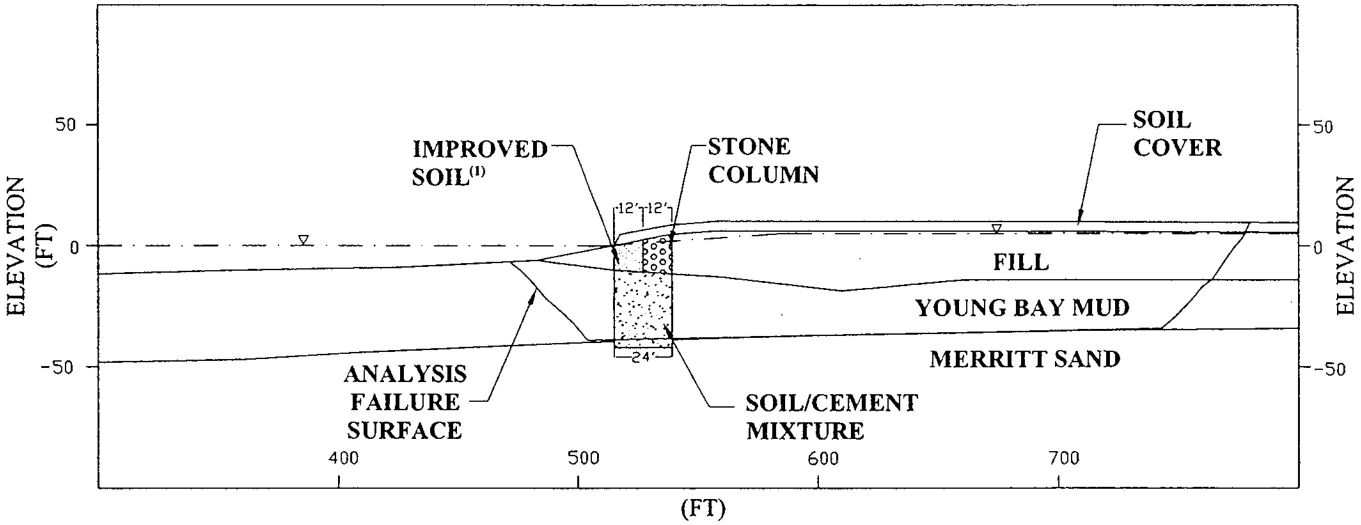
STABILITY ANALYSIS RESULTS FOR ALTERNATIVE 3 (SHEET PILES WITH ANCHORS)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

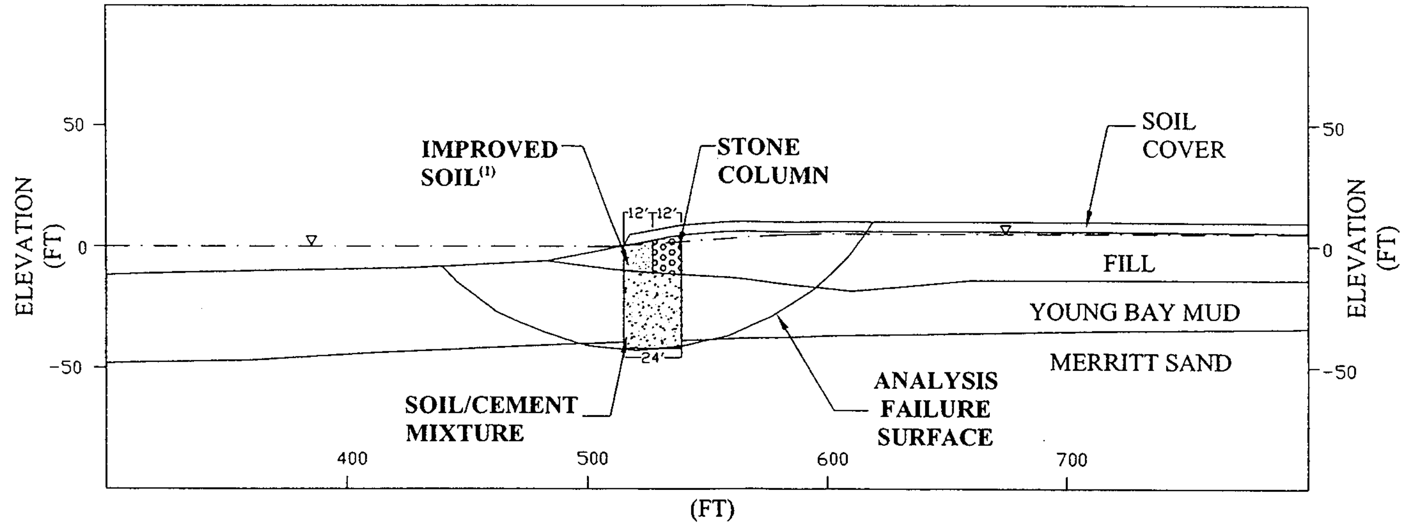
DRAWING NO: 0313684-65.DWG
DCN: FWSD-RAC-031368
CTO: # 0054
APPROVED BY: AL
CHECKED BY: TL
REV: REVISION 0
DRAWN BY: MD
DATE: 09/26/03

I:\1990-RAC\CTO-0054\DWG\031368\0313684-65.DWG
PLOT/UPDATE: SEP 15 2003 11:10:54



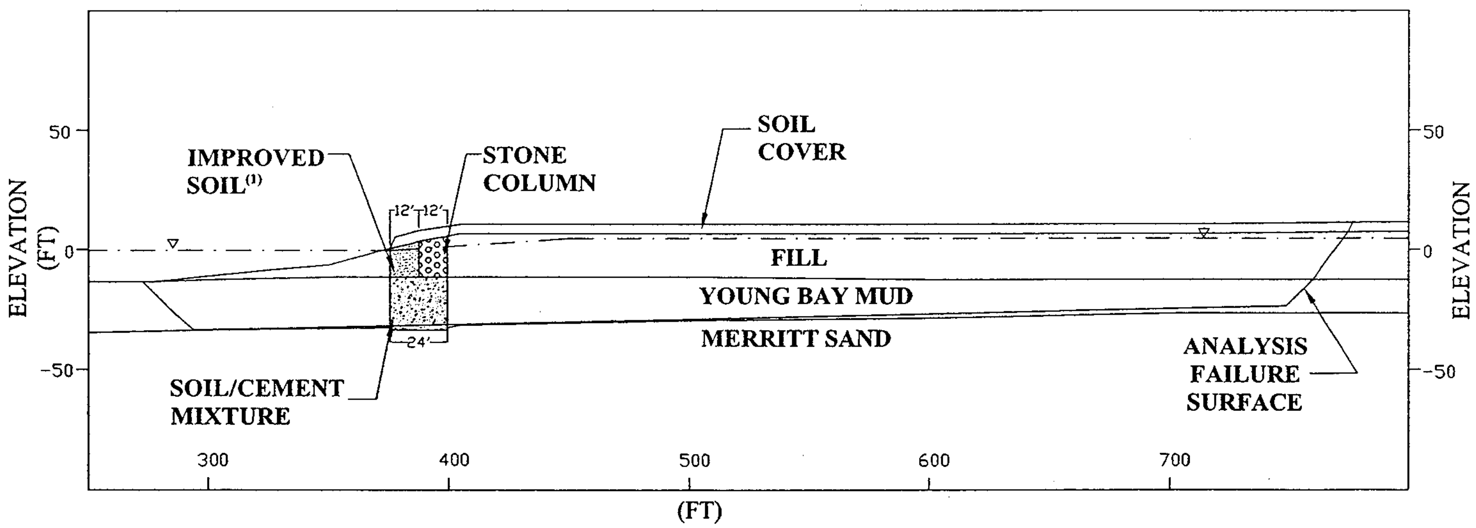
SECTION D-D'

Spencer Method, Dynamic, $K_y = 0.15$ g[(2)]



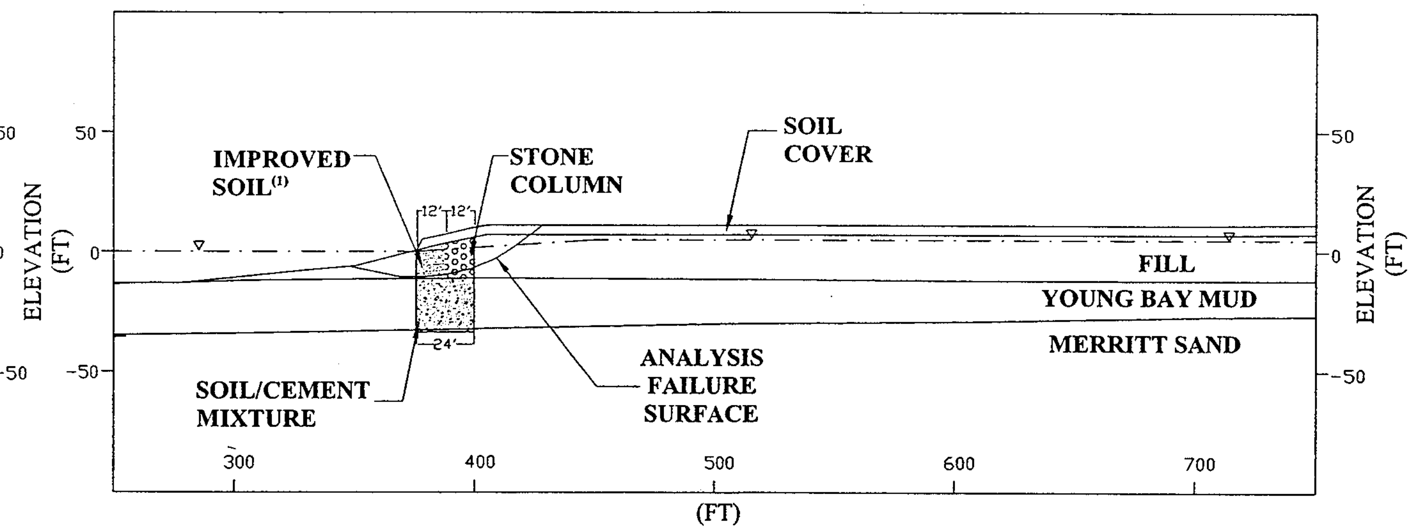
SECTION D-D'

Bishop Method, Post Earthquake, Static **F.S. = 2.13**[(2)]



SECTION F-F'

Spencer Method, Dynamic, $K_y = .15$g[(2)]



SECTION F-F'

Bishop Method, Post-Earthquake, Static **F.S. = 2.31**[(2)]

0 80' 120' 160'

SCALE
1" = 80'

Notes:

(1) Improved soil zone models the densified/improved soil matrix within the stone column area.

(2) The above stability analyses present the site stability evaluation following the implementation of remedial alternatives and include the effect of a future 4-foot soil cover.

Figure 4–66

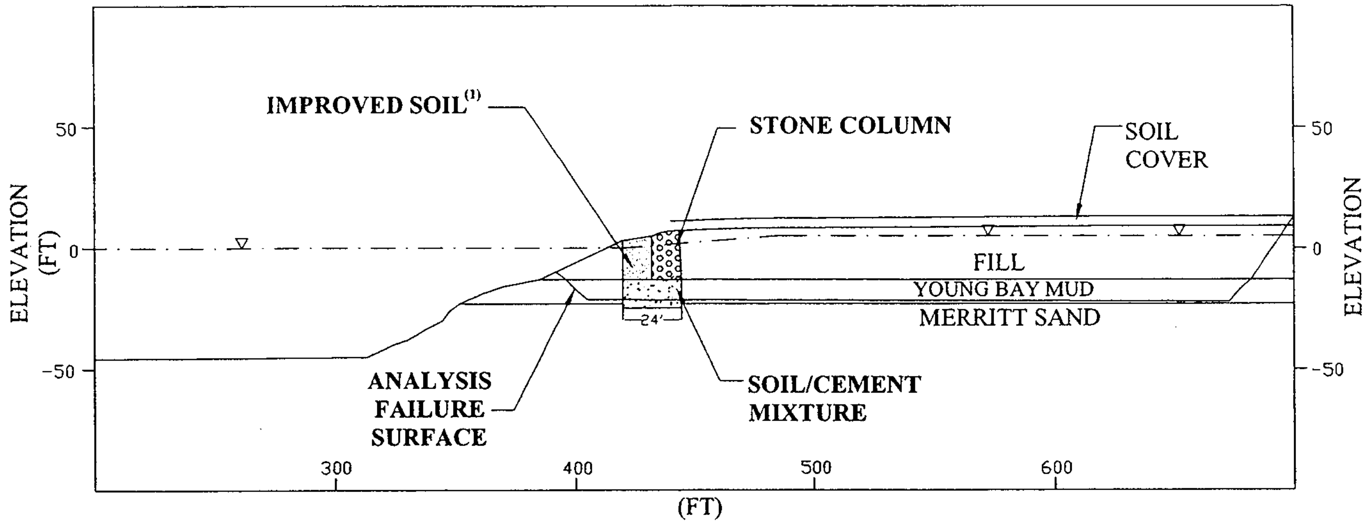
STABILITY ANALYSIS RESULTS FOR ALTERNATIVE 5 (SOIL CEMENT GRAVITY WALL AND STONE COLUMNS – SECTIONS D-D' AND F-F')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWING NO: 0313684-66.DWG
DCN: FWSD-RAC-03-1368
CTO: # 0054
APPROVED BY: AL
CHECKED BY: TL
REV: REVISION 0
DRAWN BY: MD
DATE: 09/26/03

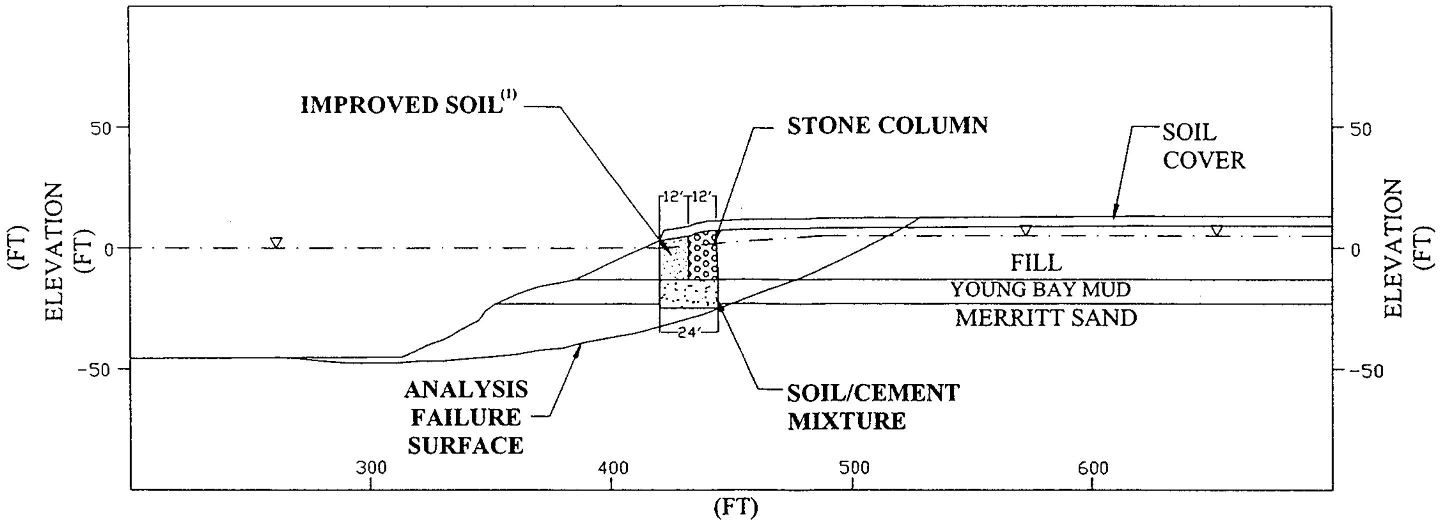
I:\1990-RAC\CTO-0054\DWG\031368\0313684-66.DWG
PLOT/UPDATE: SEP 15 2003 11:12:07



SECTION G-G'

BLOCK FAILURE PLANE

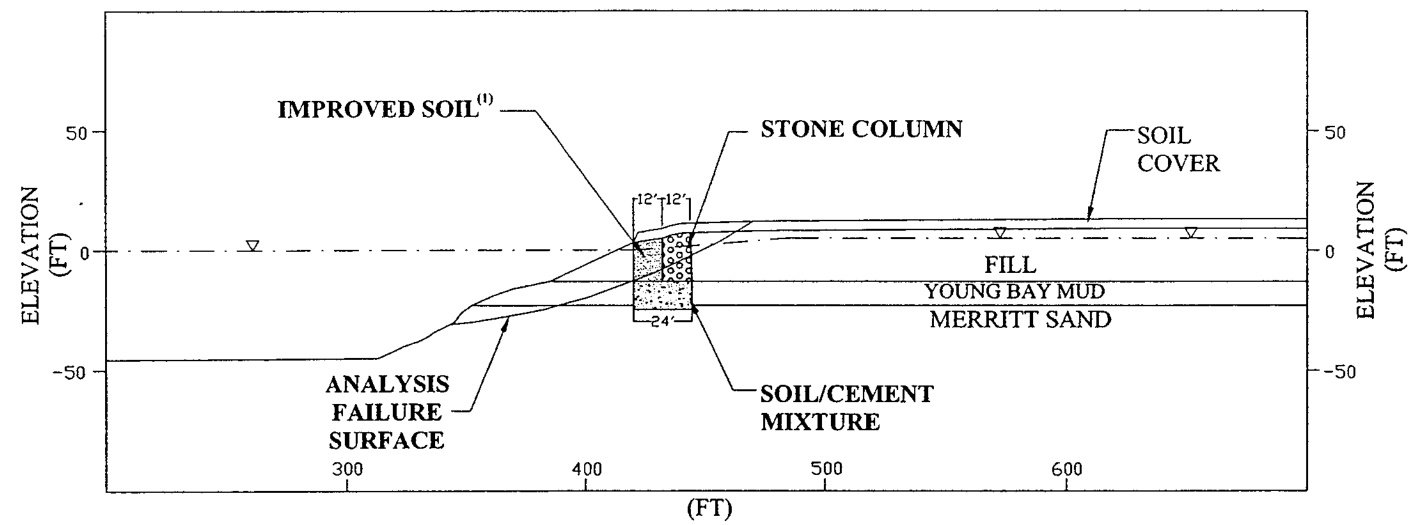
Spencer Method, Dynamic, **K_y = .19g** (2)



SECTION G-G'

CIRCULAR FAILURE PLANE

Bishop Method, Dynamic, **K_y = 0.18 g** (2)



SECTION G-G'

Bishop Method, Post-Earthquake, Static **F.S. = 1.69** (2)

0 80' 120' 160'

SCALE
1" = 80'

Notes:

(1) Improved soil zone models the densified/improved soil matrix within the stone column area.

(2) The above stability analyses present the site stability evaluation following the implementation of remedial alternatives and include the effect of a future 4-foot soil cover.

Figure 4-67

STABILITY ANALYSIS RESULTS FOR ALTERNATIVE 5 (SOIL CEMENT GRAVITY WALL AND STONE COLUMNS – SECTION G-G')

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWING NO: 0313684-67.DWG
DCN: FWSD-RAC-03-1368
CTO: # 0054
APPROVED BY: AL
CHECKED BY: TL
REV: REVISION 0
DRAWN BY: MD
DATE: 09/26/03

I:\1990-RAC\CTO-0054\DWG\031368\0313684-67.DWG
PLOT/UPDATE: SEP 15 2003 11:12:49

Global/Overall (Slope) Stability Analysis. Global/overall slope stability analysis results are presented in Figure 4-68 and summarized in Table 4-2. An estimated minimum yield acceleration of 0.23g results in minimal seismic deformations along the potential failure surface (see Figure 4-61). Computed static factors of safety for long-term and post-earthquake conditions are greater than 1.5 and 1.0, respectively.

Internal/Localized Stability Analysis. The concrete gravity wall is a massive structure subjected to similar lateral pressures on both sides of the wall, which makes this configuration stable under static and seismic loading conditions. Because of the relatively small slenderness ratio, this wall is not subjected to flexion.

This alternative is technically feasible and further analysis is warranted. Analysis results are included in Appendix A6.

Alternative 7: Excavation with Riprap

Slope excavation with riprap replacement would include a 38-foot-wide riprap wall similar to the stone column wall previously analyzed. The riprap would be installed along the shoreline as shown in Figure 4-43. Typical profiles (Sections D-D', E-E', F-F', G-G', and H-H') were shown in Figures 4-44 through 4-48. The riprap wall should be excavated into the Merritt Sand along the shoreline to prevent the development of shallower failure surfaces through the Young Bay Mud as illustrated in Figure 4-69.

Global/Overall (Slope) Stability Analysis. Global/overall slope stability analysis results are presented in Figure 4-69 and summarized in Table 4-2. A computed minimum yield acceleration of 0.12g results in seismic deformations of about 3.4 feet, which is below the performance criteria of 4 feet. This case is similar to the stone column alternative since stone column material and riprap have similar strength parameters. This alternative is seismically unstable if failure through the Young Bay Mud below the bottom of the riprap is allowed.

Internal/Localized Stability Analysis. The riprap gravity wall is a massive structure subjected to similar lateral pressures on both sides of the wall, which makes this configuration stable under static and seismic loading. Because of the relatively small slenderness ratio, this wall is not subjected to flexion.

This alternative is technically feasible, but it is similar to the stone column remedial alternative with the exception of several construction-related disadvantages. The construction of the riprap involves removal of very soft Young Bay Mud sediments from the bay floor under water, which may result in localized or large slope failures. Therefore, this alternative was not retained for further consideration. Analysis results are included in Appendix A7.

Alternative 8: Drilled Concrete Piers with Stone Columns

Drilled concrete piers could be arranged along the shoreline as shown in Figure 4-49. Two rows of staggered concrete caissons are required. Typical profiles for Sections D-D', E-E', F-F', G-G', and H-H' are presented in Figures 4-50 through 4-54. The diameter of individual piers and typical spacing used in the analysis are 3 and 8 feet, respectively. Analysis results are included in Appendix A8.

Global/Overall (Slope) Stability Analysis. Global/overall slope stability analysis results for Section D-D' are shown in Figure 4-70 and summarized in Table 4-2. For analysis, the concrete material was assumed to have a cohesion value of 144 kips per square foot. As shown in Figure 4-61, a yield acceleration value of 0.11g results in 2.6 feet of deformation under seismic loading. Pre- and post-earthquake static factors of safety meet the design criteria of 1.5 and 1.0, respectively.

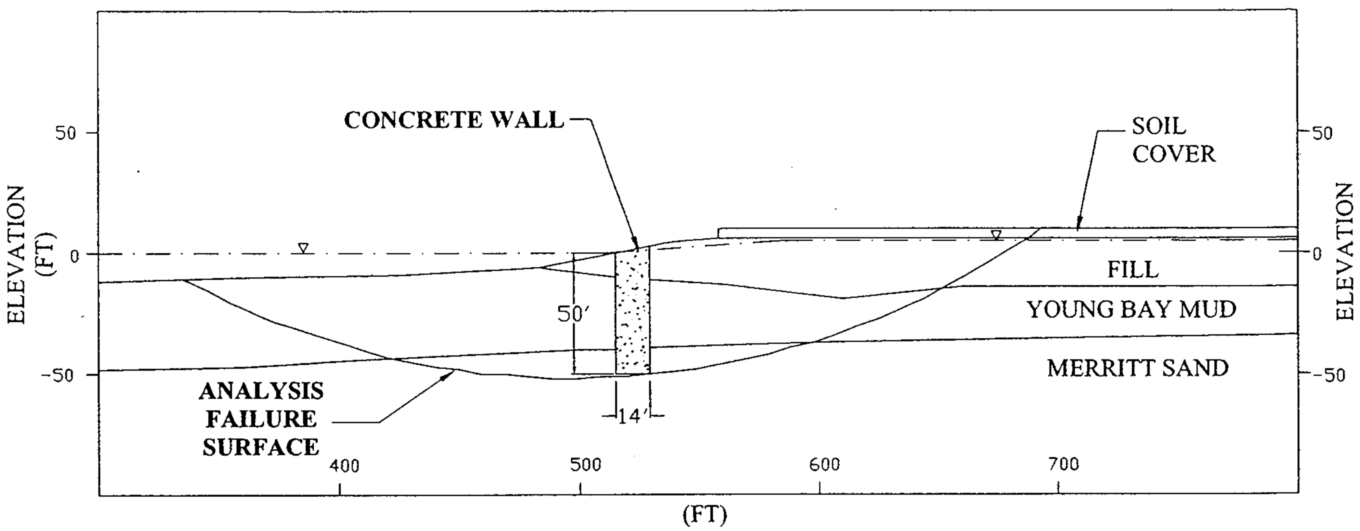
Internal/Localized Stability Analysis. Computed lateral deflections using LPILE computer program indicated a maximum deflection of 0.3 inches for cantilevered/post-earthquake static loading condition (Section D-D') at the top of the concrete pier is adequate for permanent structures. For the case of seismic loading, the pier should be designed to withstand 31 inches of deflection. Further analysis is required during the design phase to optimize pier dimensions and properties. This alternative is considered implementable, and therefore, is recommended for further evaluations.

Alternative 9: Pre-cast Concrete Piles

Pre-cast concrete piles could be arranged along the shoreline as shown in Figure 4-55. Four rows of staggered concrete piles are required. Typical profiles for Sections D-D', E-E', F-F', G-G', and H-H' are presented in Figures 4-56 through 4-60. Diameter of individual piles and typical spacing used in the analysis are 2 and 6 feet, respectively. Typical spacing perpendicular to the shoreline is 4 feet. Piles are staggered and are assumed to be driven at least 20 feet into the Merritt Sand.

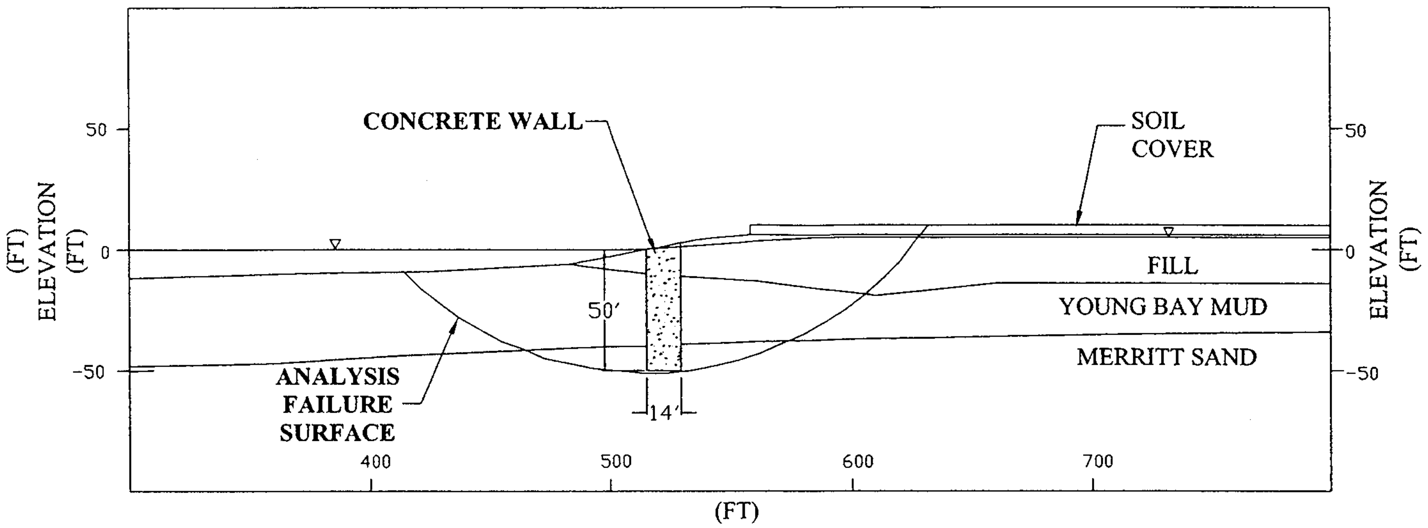
Global/Overall (Slope) Stability Analysis. Global/overall slope stability analysis results for Section D-D' for Alternative 8, drilled concrete piers are applicable for this case. Global/overall slope stability analysis results for Section D-D' are shown in Figure 4-61 and summarized in Table 4-2. For analysis, the concrete material was assumed to have a cohesion value of 144 kips per square foot. As shown in Figure 4-70, a yield acceleration value of 0.16g results in 1.9 feet of deformation under seismic loading. Long-term and post-earthquake static factors of safety meet the design criteria of 1.5 and 1.0, respectively.

Internal/Localized Stability Analysis. Computed lateral deflections using LPILE computer program indicated that a maximum deflection of 2.0 inches for cantilever/post-earthquake static loading condition (Section D-D') at the top of the driven pre-cast concrete pile is adequate for a



SECTION D-D'

Spencer Method, Dynamic, **K**y = .23g [1]



SECTION D-D'

Bishop Method, Post-Earthquake, Static F.S. = 3.17 [1]

0 80' 120' 160'

SCALE
1" = 80'

Notes:

(1) The above stability analyses present the site stability evaluation following the implementation of remedial alternatives and include the effect of a future 4-foot soil cover.

Figure 4–68

STABILITY ANALYSIS RESULTS FOR ALTERNATIVE 6 (CONCRETE WALL)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWING NO: 0313684-68.DWG

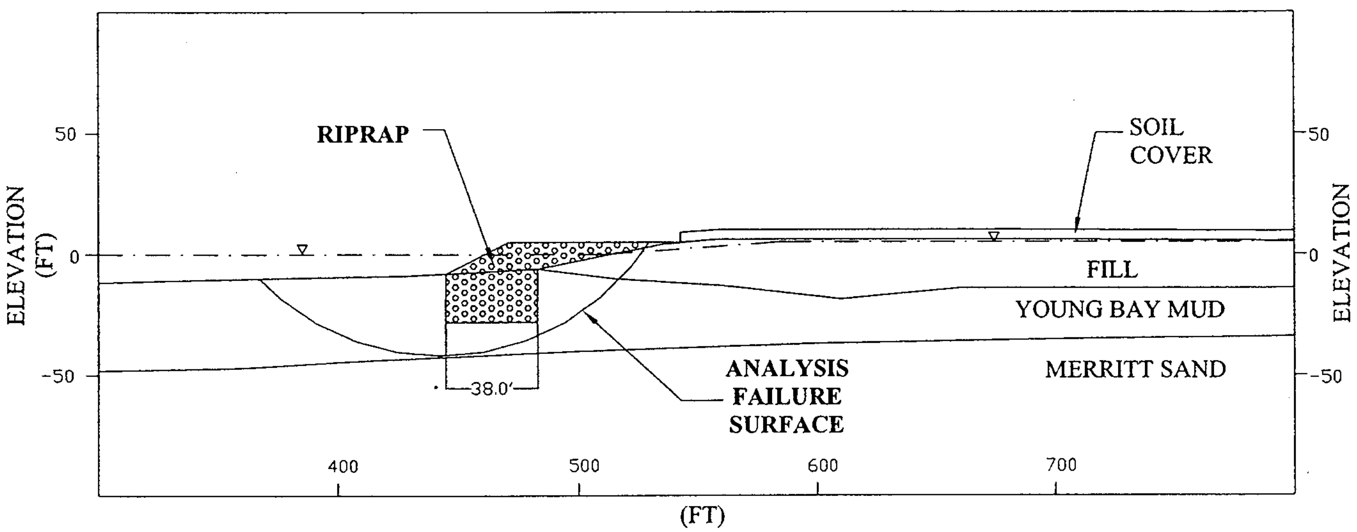
DCN: FWSD-RAC-03-1368 | CTO: # 0054

APPROVED BY: AL

CHECKED BY: TL | REV: REVISION 0

DRAWN BY: MD | DATE: 09/26/03

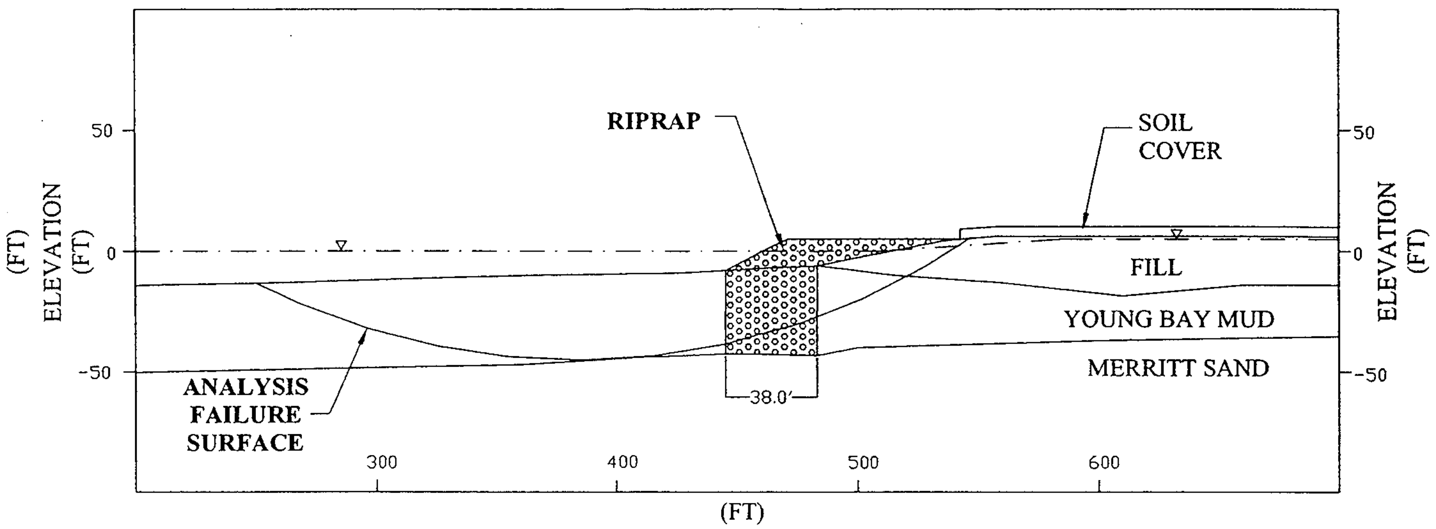
I:\1990-RAC\CTO-0054\DWG\031368\0313684-68.DWG
PLOT/UPDATE: SEP 15 2003 11:13:49



SECTION D-D'

20 ft Deep RipRap

Spencer Method, Dynamic, **K_y=.03g** [1]



SECTION D-D'

40 ft Deep RipRap

Spencer Method, Dynamic, **K_y=.12g** [1]

0 80' 120' 160'

SCALE
1" = 80'

Notes:

(1) The above stability analyses present the site stability evaluation following the implementation of remedial alternatives and include the effect of a future 4-foot soil cover.

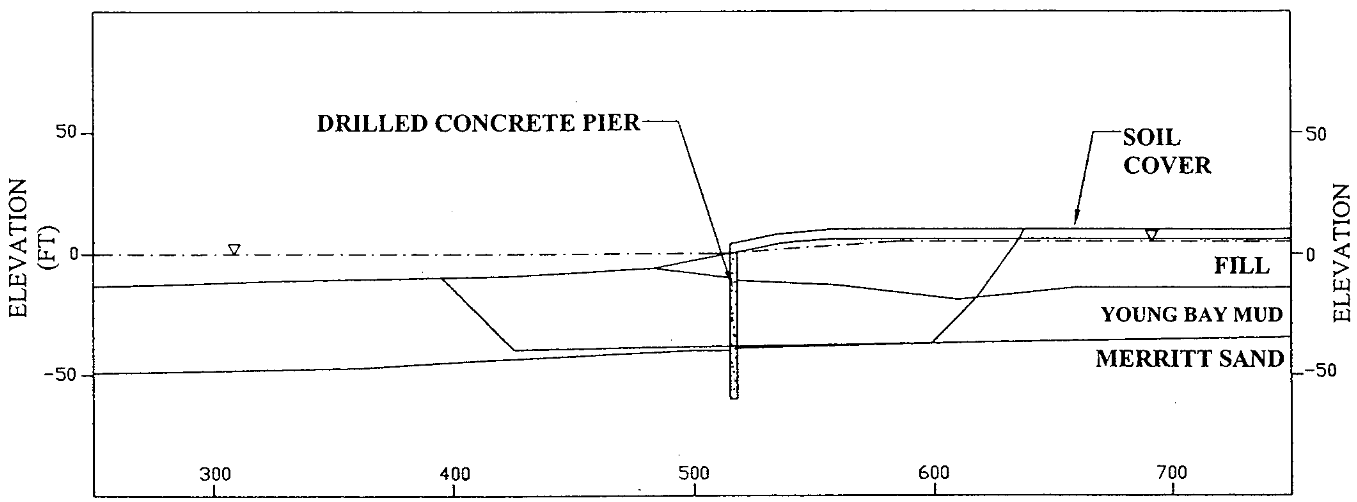
Figure 4–69
STABILITY ANALYSIS RESULTS FOR ALTERNATIVE 7
(EXCAVATION WITH RIPRAP)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWING NO: 0313684-69.DWG
DCN: FWSD-RAC-03-1368
CTO: # 0054
APPROVED BY: AL
CHECKED BY: TL
REV: REVISION 0
DRAWN BY: MD
DATE: 09/26/03

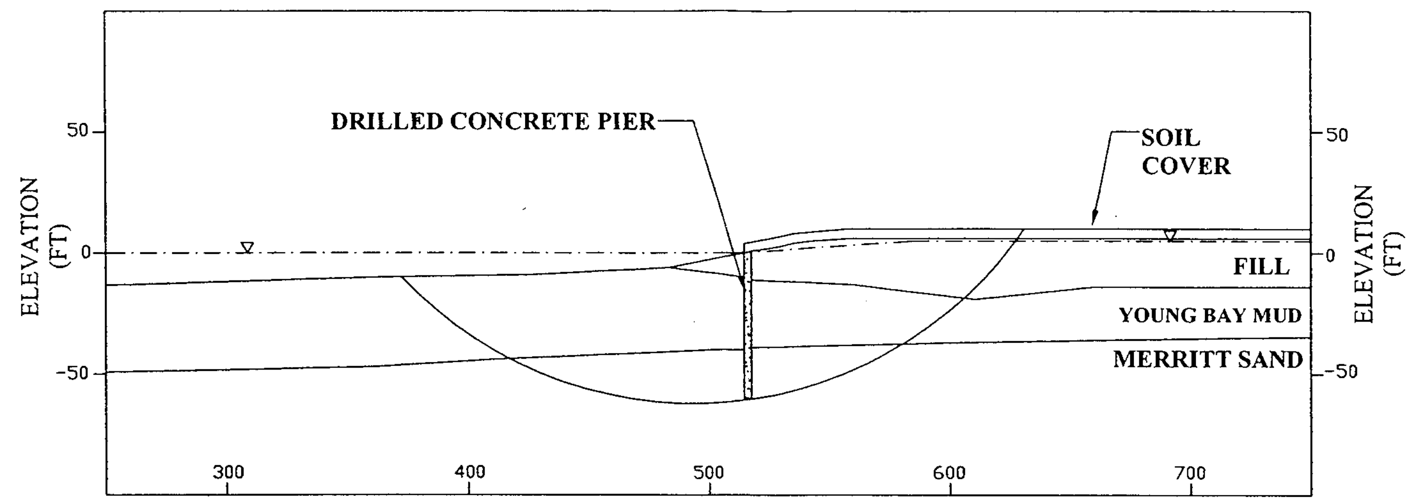
I:\1990-RAC\CTO-0054\DWG\031368\0313684-69.DWG
PLOT/UPDATE: SEP 15 2003 11:14:45



SECTION D-D'

DRILLED PIERS

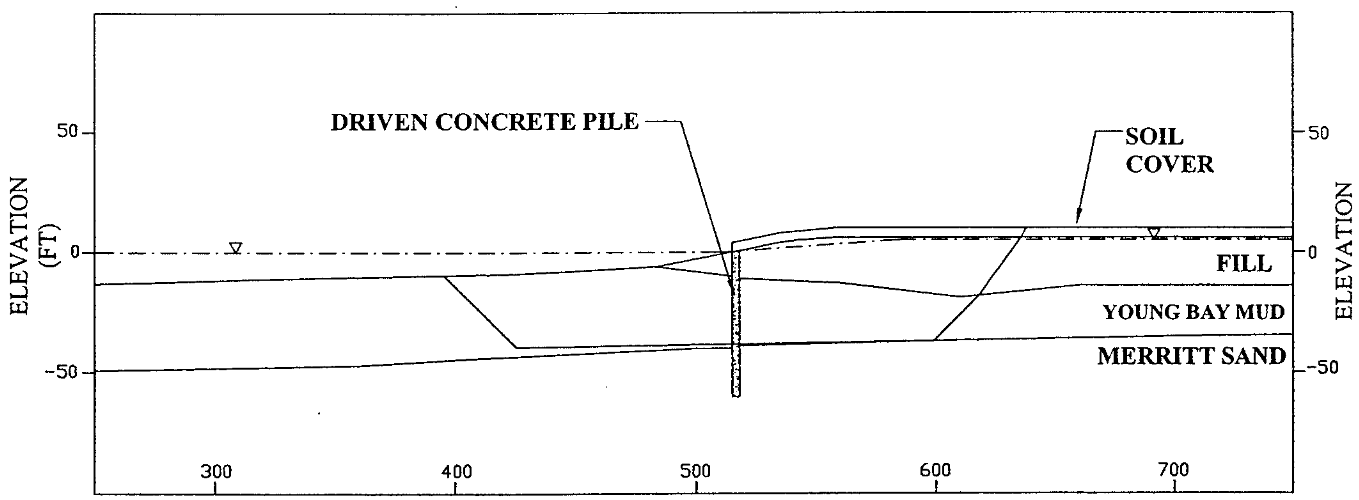
Spencer Method, Dynamic, **K_y = 0.14 g** [1]



SECTION D-D'

DRILLED PIERS

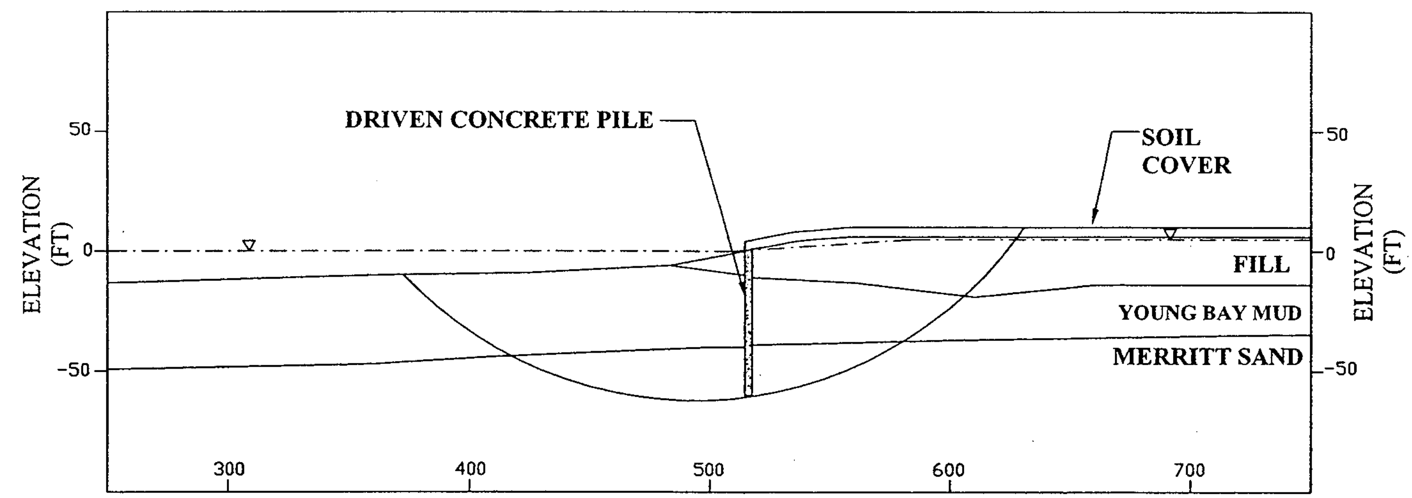
Bishop Method, Post-Earthquake, **F.S. = 4.13** [1]



SECTION D-D'

DRIVEN PILES

Spencer Method, Dynamic, **K_y = 0.16 g** [1]



SECTION D-D'

DRIVEN PILES

Bishop Method, Post-Earthquake, **F.S. = 4.13** [1]

0 80' 120' 160'

SCALE
1" = 80'

Notes:

(1) The above stability analyses present the site stability evaluation following the implementation of remedial alternatives and include the effect of a future 4-foot soil cover.

Figure 4-70

STABILITY ANALYSIS RESULTS FOR ALTERNATIVE 8 AND 9 (DRILLED OR DRIVEN CONCRETE PIERS OR PILES)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWING NO: 0313684-70.DWG
DCN: FWSD-RAC-03-1368
CTO: # 0054
APPROVED BY: AL
CHECKED BY: TL
REV: REVISION 0
DRAWN BY: MD
DATE: 09/26/03

I:\1990-RAC\CTO-0054\DWG\031368\0313684-70.DWG
PLOT/UPDATE: SEP 15 2003 11:17:20

permanent structure. For the case of seismic loading, the pile should be designed to withstand 23 inches of deflection. Further analysis is required during the design phase to optimize pile dimensions and properties. Analysis results are included in Appendix A9.

4.2.5 Technical Analysis Conclusions

The following alternatives are not considered technically feasible because of constructability concerns or they do not meet the performance criteria:

- **Wick Drains with Surcharge (Alternative 1)**

  Slopes are statically unstable (factor of safety < 1.15) under the application of the required 18-foot-high surcharge during pre-loading.

- **Sheet Piles with Anchors (Alternative 3)**

  Lateral displacements at the top of the sheet piles are too large under cantilever condition, and anchor forces are too large when sheet piles are restrained (anchored) at the top. Development of the required lateral resistance would involve very close anchor spacing plus excavation along the shoreline in the landfill area.

- **Excavation with Riprap (Alternative 7)**

  The riprap section should be extended vertically below the Young Bay Mud into the Merritt Sand Formation. This construction feature involves underwater excavation and relocation of disturbed bay sediments. Underwater excavations are likely to result in unstable conditions during implementation of the remedial measure.

Technical evaluation of the proposed remedial alternatives was performed using standard engineering methods and practices to analyze stability of the site slopes (Global/Overall Slope Stability Analysis) and structural integrity of the physical buttresses (Internal/Localized Stability Analysis). These methods provide sufficient accuracy for a feasibility-level evaluation and selection of a preferred alternative or a combination of alternatives. However, these methods do not provide a rigorous model of the geometry and material characteristics of the site slopes and structural elements of the proposed remedial alternatives. More sophisticated analysis methods are needed to evaluate the selected remedial alternative(s) at the design stage. In particular, the effects of the following factors on the performance and design of the selected remedial alternative(s) should be fully investigated:

- Two-dimensional effects of the site slopes geometry on the design earthquake ground motion
- Three-dimensional effects in the vicinity of the site northwest corner
- Nonlinear properties of the site soils, pore pressure effects, and large reduction of the site soils strength properties under strong ground shaking (for example, liquefaction of sands, strength loss of sensitive clays, and so forth)
- Dynamic soil-structure interaction effects

The above factors can be analyzed using a dynamic nonlinear finite difference or finite element numerical model, using appropriate constitutive relations to model response of the site soils and remedial alternatives structural elements.

4.2.6 Cost Evaluation

This section includes the cost evaluation of six alternatives selected based on implementability analysis described in Section 4.2.1. The evaluation involved preparing cost estimates in accordance with the U.S. Environmental Protection Agency (EPA) document entitled The Role of Cost in the Superfund Remedy Selection Process (EPA, 1996).

The types of costs that are addressed include: capital costs, operation and maintenance (O&M) costs, and present value of O&M costs. Capital costs consist of direct and indirect costs. Direct costs include expenditures for the equipment, labor, and materials necessary to install remedial alternatives. Indirect costs include expenditures for engineering, administrative, and other services required to complete the implementation of remedial alternatives. Annual O&M costs include auxiliary materials, disposal of residues, purchased services, administrative costs, maintenance reserve and contingency funds, rehabilitation costs, and costs for long-term monitoring.

This assessment evaluates the costs of the remedial alternatives on the basis of present worth. Present worth analysis allows remedial actions to be compared on the basis of a single cost representing an amount that, if invested in the base year and disbursed as needed, would be sufficient to cover all costs associated with the remedial action over its planned life. A required operating performance period and a discount rate are assumed to calculate present worth cost. A discount rate of 3.9 percent is assumed for a base calculation. This discount rate of 3.9 percent is the current interest rate for federal projects over 29 years as referenced from the Office of Management and Budget (OMB) Circular No. A-94 (OMB, 2003). The discount rate represents the anticipated difference between the rate of investment return and inflation. The present value O&M costs are included in Appendix B. The estimated costs provided for the remedial actions have an accuracy of -30 to +50 percent.

The cost estimates have been developed by analyzing the scope of work for each of the six alternatives, quantifying the work required, and developing unit rates for work, which is required. Unit rates are developed based on budgetary subcontractor pricing, past estimates/ project pricing, published production/cost database information and FWENC experience. A cost estimate report, which is included in Appendix B, has been developed with a detailed breakdown of the anticipated costs for completion of each alternative analyzed.

The cost estimate was prepared using the Hard Dollar (Grantlum Corporation, 2002) estimating software that provides:

1. The total cost for each line item with marked up unit prices
2. A summary of line item breakdown costs and markup including a summary of subactivities for each line item
3. A detailed breakdown of line item costs by each subactivity; the breakdown cost includes costs for labor, equipment, materials, subcontractors, and other miscellaneous costs

The cost estimate includes the following major line items. The scope of work or basis estimate for each line item is presented below.

Engineering – Design

All six alternatives include an engineering design cost. This cost includes a field investigation to assess site conditions, preparation of a design report, technical specifications, and construction drawings. The engineering design period is expected to last 4 months. The engineering design estimated costs for each alternative are included in Table 4-4.

Pre-Construction Costs

All six alternatives include pre-construction costs. This cost includes permitting, as well as a draft and final Work Plan, Health and Safety Plan, Erosion and Sediment Control Plan, and Construction Schedule. The pre-construction period is expected to last 2 months. Pre-construction costs for each alternative are included in Table 4-4.

Mobilization

All six alternatives include mobilization costs. This cost includes site setup and preparation, mobilization of construction equipment and personnel to the site, and perimeter fencing. Mobilization is expected to take 2 weeks to complete. The mobilization costs for each alternative are included in Table 4-4.

Stone Column Installation

Alternatives 2, 4, 5, and 8 include costs for stone column installation. This cost includes all equipment, materials, and labor necessary for the installation of stone columns using a drilling technique. The columns would consist of compacted stones with a 3-foot diameter and spaced approximately every 7 feet center to center. The stone columns would be installed from the bottom of the Young Bay Mud layer to the bottom of the surcharge for Alternatives 2 and 4, and from the bottom to the top of the fill layer for Alternatives 5 and 8. The average height of the columns is approximately 30 feet for Alternatives 2 and 4, and 20 feet for Alternatives 5 and 8. Installation is expected to take four crews 24 months to complete. The stone column installation costs for Alternatives 2, 4, 5, and 8 are included in Table 4-4.

TABLE 4-4

SUMMARY OF ESTIMATED COSTS

	Alternative 2 Stone Columns with Surcharge	Alternative 4 Stone Columns with Surcharge and Sheet Piles	Alternative 5 Soil Cement Gravity Wall and Stone Columns	Alternative 6 Concrete Wall	Alternative 8 Drilled Concrete Piers with Stone Columns	Alternative 9 Pre-cast Concrete Piles
Engineering - Design	$150,000	$244,000	$219,000	$175,000	$175,000	$175,000
Pre-construction Work	$54,000	$54,000	$54,000	$54,000	$54,000	$54,000
Mobilization	$64,000	$89,000	$76,000	$64,000	$118,000	$93,000
Stone Column Installation	$14,100,000	$14,100,000	$2,914,000	N/A	$752,000	N/A
Soil Cement Mixing	N/A	N/A	$8,550,000	N/A	N/A	N/A
Concrete Wall	N/A	N/A	N/A	$8,480,000	N/A	N/A
Concrete Piers	N/A	N/A	N/A	N/A	$8,424,000	N/A
Pre-cast Concrete Piles	N/A	N/A	N/A	N/A	N/A	$13,122,000
Surcharge Placement	$9,612,000	$2,682,000	N/A	$N/A	N/A	N/A
Contaminated Soil Placement	$640,000	$640,000	$141,000	$1,600,000	$141,000	N/A
Slide Rail System	N/A	N/A	N/A	$450,000	N/A	N/A
Waterloo Sheet Piling	N/A	$7,200,000	N/A	N/A	N/A	N/A
Sheet Pile Anchors	N/A	$800,000	N/A	N/A	N/A	N/A
Anchor Excavation/Backfill	N/A	N/A	N/A	N/A	N/A	N/A
Water Treatment System	$210,000	$210,000	$93,000	$170,000	$93,000	N/A
Hydro-seeding Restoration	$20,000	$20,000	$4,000	$20,000	$20,000	$10,000
Engineering Oversight	$504,000	$504,000	$252,000	$189,000	$231,000	$231,000
Construction Project Support	$2,640,000	$2,640,000	$1,368,000	$1,053,000	$1,265,000	$1,265,000
Demobilization	$26,000	$26,000	$19,000	$19,000	$67,000	$42,000
Total Capital Costs	**$28,020,000**	**$29,209,000**	**$13,690,100**	**$12,274,000**	**$11,340,100**	**$14,992,000**
Total Operation and Maintenance (O&M) Costs for 30 years	**$1,011,590**	**$1,011,590**	**$183,930**	**$183,930**	**$183,930**	**$183,930**
Total Project Costs (Capital and O&M)	**$29,031,590**	**$30,220,590**	**$13,874,030**	**$12,457,930**	**$11,524,030**	**$15,175,930**

Notes:

N/A – not applicable

Soil Cement Mixing

Alternative 5 includes the installation of the soil cement gravity wall. This cost includes all equipment, materials, and labor necessary for the injection of a stabilizing agent such as cement slurry to mix with the Young Bay Mud material. The area to be stabilized is only within the Young Bay Mud and is approximately 24 feet wide by approximately 4,000 feet long, with an average depth of 23 feet. Installation is expected to take three crews 12 months to complete. The soil cement mixing costs are included in Table 4-4.

Concrete Wall

Alternative 6 includes cost for a concrete wall. This cost includes all equipment, materials, and labor necessary for the installation of a 14-foot-wide and 35-foot-deep concrete wall, which extends approximately 4,000 feet. Installation is expected to take 9 months. The concrete wall costs are included in Table 4-4.

Drilled Concrete Piers

Alternative 8 includes cost for installation of a system of drilled concrete piers. This cost includes all equipment, materials, and labor necessary for the installation of two rows of 5,000-psi concrete piers. The concrete piers will be installed in locations as shown in Figures 4-49 through 4-54. The pier would be 3 feet in diameter and spaced 8 feet center to center. The piers would be installed from the ground surface to 60 feet deep. Installation is expected to take four crews 11 months to complete. The concrete pier costs are included in Table 4-4.

Pre-cast Concrete Piles

Alternative 9 includes cost for pre-cast concrete pile installation. This cost includes equipment, materials, and labor necessary for installation of four rows of 5,000 psi pre-cast concrete piles. These piles would be installed at locations as shown in Figures 4-55 through 4-60. The pre-cast concrete piles would be 2 feet in diameter and spaced 6 feet center to center. They would be installed from the ground surface to 60 feet deep. The pre-cast concrete piles costs are included in Table 4-4.

Surcharge Placement

Alternatives 2 and 4 include fill surcharge placement costs. This cost includes all equipment, materials, and labor necessary for the placement of approximately 101,333 cubic yards in Alternative 2 and 14,815 cubic yards in Alternative 4 of surcharge material. After full consolidation of the Young Bay Mud layer, the lower 4 feet of the fill surcharge will remain in place as part of the landfill cap, and the remaining fill material will be used as additional fill required for future grading operations associated with golf course construction. The cost of the surcharge placement shown in Table 4-4 includes the cost for the placement of the surcharge for the full consolidation of the Bay Mud layer. This cost is based on placement of the 4-foot-thick landfill cap only within the limits of the area where surcharge fill would be placed. The surcharge placement costs of the two alternatives are included in Table 4-4.

Contaminated Soil Placement

Alternatives 2, 4, 5, 6, and 8 include the excavation of contaminated soil and Young Bay Mud. During the installation of the stone columns, the concrete wall, drilled concrete piers, both contaminated soil and Young Bay Mud will be excavated. This cost includes all equipment, materials, and labor necessary for the excavation, placement of the contaminated soils in the on-site landfill, and 2 feet of fill material from an off-site source to temporarily cap the excavated contaminated soil in the landfill area. The excavated soil placement costs for each alternative are included in Table 4-4.

Slide Rail System

Alternative 6 includes costs for a slide rail system to excavate a 14-foot-wide trench. This cost includes all equipment, materials, and labor necessary for the slide rail trench support system. The slide rail system cost is included in Table 4-4.

Sheet Piling

Alternative 4 includes installation costs for sheet piling. This cost includes the equipment and personnel necessary for the installation of sheet piling with watertight joints and low permeability grout, as well as construction quality control personnel. The sheet piles will be driven from the ground surface to approximately 60 feet deep with a length of approximately 4,000 feet. Installation is expected to take two crews 6 months to complete. The sheet piling cost is included in Table 4-4.

Sheet Pile Anchors

Alternative 4 includes sheet pile anchor costs. This cost includes the equipment, materials, and labor necessary for the wall anchors for the Waterloo sheet piling. The wall anchor cost is included in Table 4-4.

Water Treatment System

Alternatives 2, 4, 5, 6, and 8 include water treatment systems. This cost includes all equipment, materials, and labor necessary for the design, installation, and operation of a temporary dewatering and treatment system. The water treatment system costs for each alternative are included in Table 4-4.

Hydro-seeding Restoration

All six alternatives include a cost for hydro-seeding restoration. This cost includes all equipment, material, and labor necessary for the vegetative restoration of areas disturbed by construction activities. The hydro-seeding restoration costs for each alternative is included in Table 4-4.

Engineering Oversight

All six alternatives include engineering oversight cost. This cost includes labor and per diem costs for personnel to oversee construction activities. This cost is dependent on the duration of the work. The engineering oversight costs for each alternative are included in Table 4-4.

Construction Project Support

All six alternatives include a construction project support cost. This cost includes home office support to maintain the ongoing project and supplies associated with field and office operations. Costs include heavy and light equipment and tools, personal protective equipment, temporary facilities and supplies, utilities, decontamination of equipment, testing laboratory, surveying, health and safety training, audits and inspections, and other miscellaneous costs. This cost is dependent on the duration of the work. The construction project support costs for each alternative are included in Table 4-4.

Demobilization

All six alternatives include a demobilization cost. This cost includes final site cleanup, demobilization of construction equipment and personnel, and demobilization of facilities. Demobilization is expected to take 2 weeks to complete. The demobilization costs for each alternative are included in Table 4-4.

Operation and Maintenance

The O&M costs for all six alternatives are included in Table 4-4. These costs are the total O&M costs to maintain the remedial alternative for 30 years.

The O&M cost for Alternatives 2 and 4 would individually be $682,470. The cost includes all equipment, materials, and labor necessary to perform the following:

- Install temporary water treatment system.
- Treat collected water from stone column.
- Change water treatment system filters.
- Collect and perform analytical testing on water samples.
- Repair surcharge to address soil erosion.

- Perform semiannual inspection.
- Prepare semiannual report.

The O&M cost for the Alternatives 5, 6, 8, and 9 would individually be $124,090. The cost includes all equipment, materials, and labor necessary to perform the following:

- Perform semiannual inspection.
- Prepare semiannual report.

Summary of Costs

Table 4-4 summarizes the total costs for all six alternatives. The costs for Alternatives 5, 6, 8, and 9 are in the same range (that is, $11 million to $15 million). The costs for Alternatives 2 and 4, however, are much higher. Based on high cost estimates for Alternatives 2 and 4, it was determined that these alternatives would not be considered for further detailed analysis.

4.3 COMPARATIVE ANALYSIS

A comparative analysis of the remaining alternatives (5, 6, 8, and 9) was performed to select a recommended alternative. The four remaining alternatives include Alternative 5 (soil cement gravity wall and stone columns), Alternative 6 (concrete wall), Alternative 8 (drilled concrete piers with stone columns) and Alternative 9 (pre-cast concrete piles). Based on results from the individual analysis of alternatives described in Section 4.2, all four alternatives are implementable and are relatively cost effective. Nine EPA evaluation criteria (discussed earlier in Section 3.2) were used to compare each alternative with one another. The criteria include: 1) overall protection of human health; 2) compliance with applicable or relevant and appropriate requirements (ARARs); 3) long-term effectiveness and permanence; 4) reduction of toxicity, mobility, and volume through treatment; 5) short-term effectiveness; 6) implementability; 7) cost; 8) state or support agency acceptance; and 9) community acceptance. Table 4-5 presents a summary of the results of this comparative analysis. A brief discussion of the approach used in the application of these criteria is provided below.

Overall Protection of Human Health and the Environment

The application of this criteria, involves evaluating overall performance and effectiveness of each alternative to control the potential release of waste into San Francisco Bay during a design earthquake. Release of waste was considered the key hazard associated with the protection of human health and the environment. Other concerns relevant to protecting human health and the environment are addressed in criteria 2, 3, 4, and 5 in Table 4-5.

TABLE 4-5

COMPARATIVE ANALYSIS

Evaluation Criteria #	Description	Alternative 5 Soil Cement Gravity Wall and Stone Columns	Alternative 6 Concrete Wall	Alternative 8 Drilled Concrete Piers with Stone Columns	Alternative 9 Pre-cast Concrete Piles
1	Overall Protection of Human Health	• Provides adequate protection by controlling release of waste into San Francisco Bay/Oakland Inner Harbor. The gravity wall also creates a wide buffer zone that provides additional protection.	• Provides adequate protection by controlling release of waste into San Francisco Bay/Oakland Inner Harbor.	• Provides adequate protection by controlling release of waste into San Francisco Bay/Oakland Inner Harbor.	• Provides adequate protection by controlling release of waste into San Francisco Bay/Oakland Inner Harbor.
2	Compliance with ARARs	• ARARs compliance considered adequate. • Additional ARARs pertaining to soil additives, groundwater monitoring, and hazardous waste management are also considered applicable and require compliance.	• ARARs compliance considered adequate. • Additional ARARs pertaining to soil additives and hazardous waste management are also considered applicable and require compliance. • Similar to Alternative 5, but no groundwater monitoring is required since no stone columns are involved.	• ARARs compliance considered adequate. • Additional ARARs pertaining to groundwater monitoring and hazardous waste management. • Compliance with ARARs is slightly more difficult than in other alternatives due to large amount of potentially impacted soil to be excavated, which will require compliance with another set of ARARs.	• ARARs compliance considered adequate. • No additional ARARs are identified. • Compared to Alternatives 5, 6, and 8, there is less excavation and intrusive work, which should result in easier compliance with ARARs.
3	Long-term Effectiveness and Permanence	• Fill layer in improved areas will be densified during placement of stone column, which will reduce the liquefaction potential during a design earthquake. • The gravity wall creates a wide improved soil zone that can deform under design earthquake loading without major damage. • Some maintenance/monitoring is required for the stone columns.	• The concrete wall in the fill layer will not be susceptible to damage from liquefaction potential during a design earthquake. • The concrete wall acts as a rigid system that increases its potential for some cracking due to lateral movement in the long term. • No major maintenance/monitoring is required.	• Fill layer in improved areas will be densified during placement of stone column, which will reduce the liquefaction potential during a design earthquake. • System of drilled concrete piers is more flexible than a concrete wall, but lateral movement during a design earthquake can result in damage. • Some maintenance/monitoring is required for the stone columns.	• Liquefaction potential remains high in the fill layer since no direct measures, such as stone columns or soil additives, are implemented. • System of pre-cast concrete piles is more flexible than concrete wall, but lateral movement during a design earthquake can result in damage. • Minor maintenance/monitoring is required.
4	Reduction of Toxicity, Mobility, and Volume Through Treatment	• Some concerns regarding release of impacted soil, drill cuttings, and water during construction. • Volume of potentially impacted soil excavated or released to the site is lower than Alternative 6. • Cement slurry is mixed with soil, but is not considered to impact/increase toxicity of soil.	• Relatively more concerns regarding release of impacted soil, drill cuttings, and water during construction due to deeper excavations required. • Volume of potentially impacted soil excavated or released to the site is highest compared to Alternatives 5, 8, and 9 due to deeper excavations required. • No soil additives involved with this alternative.	• Some concerns regarding release of impacted soil, drill cuttings, and water during construction. • No soil additives involved with this alternative. • Volume of potentially impacted soil excavated or released to the surface is low compared to Alternatives 5, and 6.	• Relatively less concerns regarding release of impacted drill cutting and water during construction due to limited excavations (boreholes) required. • Volume of potentially impacted soil excavated or released to the surface is lowest compared to Alternatives 5, 6, and 8. • No soil additives involved with this alternative.

TABLE 4-5 (Continued)

COMPARATIVE ANALYSIS

Evaluation Criteria #	Description	Alternative 5 Soil Cement Gravity Wall and Stone Columns	Alternative 6 Concrete Wall	Alternative 8 Drilled Concrete Piers with Stone Columns	Alternative 9 Pre-cast Concrete Piles
5	Short-term Effectiveness	• No major issues concerning performance of this alternative immediately after the construction are identified.	• No major issues concerning performance of this alternative immediately after the construction are identified.	• No major issues concerning performance of this alternative immediately after the construction are identified.	• No major issues concerning performance of this alternative immediately after the construction are identified.
6	Implementability	• Implementability considered feasible with no constructability issues. • Long-term static slope stability fs: 3.03. • Post-earthquake static slope stability fs: 2.13. • Estimated lateral slope movement: 1.9 ft. • Acceptable administrative feasibility.	• Implementability considered feasible with no constructability issues. • Long-term static slope stability fs: 3.50. • Post-earthquake static slope stability fs: 3.17. • Estimated lateral slope movement: 0.6 ft. • Acceptable administrative feasibility.	• Implementability considered feasible with no constructability issues. • Long-term static slope stability fs: 4.34 • Post-earthquake static slope stability fs: 4.06 • Estimated lateral slope movement: 3 ft. • Placement/handling of very long piers (60 ft in length) is an issue during the construction phase. • Acceptable administrative feasibility.	• Implementability considered feasible with no constructability issues. • Long-term static slope stability fs: 4.34. • Post-earthquake static slope stability fs: 4.06. • Estimated lateral slope movement: 3 ft. • Driving long piles (80 ft in length) is an issue during the construction phase because of possible contact with riprap along the shoreline perimeter. • Acceptable administrative feasibility.
7	Cost	• Capital cost: $13,690,100 • O&M cost: $183,930 • Total cost: $13,874,030	• Capital cost: $12,274,000 • O&M cost: $183,930 • Total cost: $12,457,930	• Capital cost: $11,340,100 • O&M cost: $183,930 • Total cost: $11,524,030	• Capital cost: $14,992,000 • O&M cost: $183,930 • Total cost: $15,175,930
8	State or Support Agency Acceptance	At this stage, no issue(s) have been identified that would make any alternative more or less acceptable to state or support agencies. This criterion will be addressed in more detail in a ROD after public comments for the RI/FS Report and PP are available.			
9	Community Acceptance	At this stage, no issue(s) have been identified that would make any alternative more or less acceptable to the public. This criterion will be addressed in more detail in a ROD after public comments for the RI/FS Report and PP are available.			

Notes:

ARAR – applicable or relevant and appropriate requirement
fs – factor of safety
ft – feet
O&M – operation and maintenance
ROD – Record of Decision
RI/FS – Remedial Investigation/Feasibility Study
PP – Proposed Plan

Compliance with ARARs

ARARs common to all four alternatives pertain to potential impacts to wetland located outside of the Installation Restoration (IR) Site 1 boundary, filling within navigable waters, shoreline work, excavation related to archaeological resources, impact to endangered species/habitat, stormwater control, landfill cap, and drilling permits. These ARARs are listed in Table 3-2. The evaluation process involved comparison of each alternative's ability to comply with these ARARs. Additional ARARs specifically applicable to each alternative were identified and considered in the evaluation process.

Overall Protection of Human Health and the Environment

The application of this criteria, involves evaluating overall performance and effectiveness of each alternative to control the potential release of waste into San Francisco Bay during a design earthquake. Release of waste was considered the key hazard associated with the protection of human health and the environment. Other concerns relevant to protecting human health and the environment are addressed in criteria 2, 3, 4, and 5 in Table 4-5.

Compliance with ARARs

ARARs common to all four alternatives pertain to potential impacts to wetland located outside of the Installation Restoration (IR) Site 1 boundary, filling within navigable waters, shoreline work, excavation related to archaeological resources, impact to endangered species/habitat, stormwater control, landfill cap, and drilling permits. These ARARs are listed in Table 3-2. The evaluation process involved comparison of each alternative's ability to comply with these ARARs. Additional ARARs specifically applicable to each alternative were identified and considered in the evaluation process.

Long-Term Effectiveness and Permanence

The long-term effectiveness of each alternative was compared based on its anticipated ability to maintain structural integrity over time. Performance limitations or attributes such as flexibility and rigidity of each structure were considered as part of the evaluation. Maintenance requirements were also compared to identify a favorable alternative.

Reduction of Toxicity, Mobility, and Volume through Treatment

The selected remedial alternatives are designed to control the release of waste into San Francisco Bay, without having an adverse impact on the existing site conditions. The evaluation of this criteria involved comparison of potential impacts to the site resulting from implementation of each alternative. The potential impacts to toxicity, mobility, and volume of waste are limited to construction activities and can be addressed during construction without any long-term negative effects.

Short-Term Effectiveness

Short-term effectiveness for all four alternatives is anticipated to be acceptable because the alternatives will perform adequately immediately after construction. Therefore, comparative analysis using this criterion did not result in a preference for one alternative over another.

Implementability

This criterion was used to evaluate the administrative and technical feasibility of each alternative. Based on the ARARs identified for this Geotechnical Feasibility Study (FS) Report (see Table 3-2), administrative feasibility was found to be acceptable for all four alternatives.

Technical feasibility involves meeting the performance criteria and a determination that the constructability of the alternative is practical and can be achieved. A technical analysis indicated that all four alternatives satisfy the performance criteria. Long-term static slope stability factors of safety varied from 3.03 to 4.34. Post-earthquake static slope stability factors of safety varied from 2.13 to 4. The estimated lateral slope movement varied from 0.6 to 3.0 feet. These results indicate that all four alternatives meet the performance criteria. Therefore, the comparison of alternatives for this criteria, was mainly based on the relative implementability of the standard methods of construction associated with each alternative.

Cost

The cost for each alternative was broken down into capital and O&M costs as presented in Table 4-4.

State or Support Agency Acceptance

No state or agency acceptance issues are anticipated at this stage for any of the alternatives. Therefore, comparative analysis for this criterion did not yield a favored alternative. This criteria may need to be addressed in more detail in a Record of Decision (ROD) after public comments for the RI/FS Report and Proposed Plan (PP) are available.

Community Acceptance

No community acceptance issues are anticipated at this stage for any of the alternatives; therefore, comparative analysis for this criterion did not yield a favored alternative. This criteria may need to be addressed in more detail in a ROD after public comments for the RI/FS Report and PP are available.

4.4 RECOMMENDED ALTERNATIVE

Based on the comparative analysis performed in the previous section and summarized in Table 4-5, the recommended alternative is Alternative 5, soil cement gravity wall with stone columns. The main factors or evaluation criteria favoring this alternative are overall protection of human health and the environment, long-term effectiveness and permanence, and implementability. Other criteria considered, such as compliance with ARARs, short-term effectiveness, state or support agency acceptance, and community acceptance, did not influence the selection process because of similar performance of each alternative related to these criteria. Overall cost for this alternative is higher compared to Alternatives 6 and 8. However, the anticipated superior long-term performance of Alternative 5 and relatively similar costs of the four alternatives ($11 to $15 million) were key factors in selecting this alternative as the recommended alternative to mitigate geotechnical and seismic hazards identified in the RI Report Addendum, Volume III (FWENC, 2002).

A summary of the basis for selection of the recommended alternative is presented below:

- **Overall Protection of Human Health and the Environment** – Alternative 5 is anticipated to outperform the other alternatives because of its ability to provide a better control of waste containment in a design earthquake event. This is due to the greater width of the gravity wall, which maintains adequate separation of the waste and the navigable waters more effectively.
- **Long-Term Effectiveness and Permanence** – Alternative 5 performs better than other alternatives when considering long-term performance. This is due to the improved soil conditions created by its implementation and mitigation of potential liquefaction impacts as well. The gravity wall also can deform with the design earthquake and is less susceptible to damage than the other alternatives.
- **Implementability** – Alternative 5 is relatively easier to construct than the other alternatives. The gravity wall and stone columns require no special equipment or procedures for construction.
- **Cost** – O&M costs for all four alternatives are similar. These costs only cover routine maintenance activities. Capital costs for Alternative 5 are second highest among the four alternatives. However, as discussed above, the superior long-term performance was a key factor in selecting this alternative as the recommended alternative to mitigate geotechnical and seismic hazards identified in the RI Report Addendum, Volume III (FWENC, 2002). Also, during a seismic event, Alternative 5 is expected to experience less damage than other alternatives. Therefore, if costs associated with repairs resulting from design earthquake are considered in the overall cost comparison, Alternative 5 would be more favorable from a cost comparison standpoint as well.

It is recommended that during the detailed design stage, the extent of the remedial measure (area and depth of application) should be further evaluated to determine if these can be reduced (optimized) based on the following:

- Detailed waste delineation along shoreline perimeter.
- More sophisticated detailed analysis (such as Finite Element modeling) to obtain more accurate assessment of slope movement.
- Risk assessment to determine impact of waste release into the San Francisco Bay/Oakland Inner Harbor Channel.

Also, if additional information gathered or evaluations performed during the detailed design stage demonstrate viability of other remedial actions (components) over the preferred alternative (Alternative 5), then the preferred remedy may be altered. For example, the stone cement walls could be extended from the ground surface through the bottom of the Young Bay Mud layer, replacing the stone columns in the upper fill layer. It must be emphasized that any changes made to the preferred remedial alternative must undergo the same screening process to evaluate technical implementability, cost, and long-term effectiveness. Any changes will also need to be re-approved once the preferred remedial alternative has been approved and documented in the final ROD for Operable Unit 3 following issuance of the PP and consideration of public comments.

5.0 REFERENCES

Achilleos, E. 1988. *User's Guide for PC-STABL-5M, Informational Report*. Purdue University, Lafayette, Indiana.

Bartlett, S.F. and T.L. Youd. 1992. Empirical Prediction of Liquefaction-Induced Lateral Spread. *Journal of Geotechnical Engineering, ASCE*, v. 121, No. 4, p. 316-329.

Beikae, M., M. Luebbers, J. Barneich, and D. Osmun. 1996. *Seismic Deformation Analyses of Eastside Reservoir Dams*. 16th Annual Lecture Series by United States Committee On Large Dams (USCOLD), Los Angeles, California. July.

Beikae, M. 2002. Personal Communication on Allowable Seismic Deformations for Earth Dams.

Day, R.W. 2002. *Geotechnical Earthquake Engineering Handbook*. McGraw-Hill: New York, NY.

Duncan, J.M. 1992. *State-of-the-Art: Static Stability and Deformation Analysis*. ASCE Geotechnical Special Publication No. 31, Stability and Performance of Slopes and Embankments-II. pp. 222-266.

Ecology and Environment. 1983. *Initial Assessment Study of Naval Air Station, Alameda, California, Final Report*. Prepared for Navy Assessment and Control of Installation Pollutants and Naval Energy and Environmental Support Activity, Port Hueneme, California.

Ensoft, Inc. 1999. Computer Program LPILE Plus, Version 3.0: A Program for the Analysis of Piles and Drilled Shafts Under Lateral Loads. By L. C. Reese, S. T. Wang, J. A. Arrellaga, and J. Hendrix.

Foster Wheeler Environmental Corporation (FWENC). 2001. *Final Focused Remedial Investigation Work Plan*. Ordnance and Explosives Waste Characterization, and Geotechnical and Seismic Evaluations at Installation Restoration Site 1. Alameda Point, Alameda, California.

_______________. 2002. *Final Ordnance and Explosives Waste/Geotechnical Characterization Report* (RI Addendum, Volume III). Ordnance and Explosives Waste Characterization, and Geotechnical and Seismic Evaluations at Installation Restoration Site 1, Alameda Point, Alameda, California.

Grantlum Corporation. 2002. Hard Dollar Software Version 2002. Tempe, Arizona.

Huang, Y.H. 1983. *Stability of Analysis of Earth Slopes*. Van Nostrand Reinhold Company.

Idris, I.M. and Sun. J. I. 1991. Computer Program for Conducting Equivalent Linear Seismic Response Analyses of Horizontally Layered Soil Deposits-Program modified based on the Original SHAKE program published in December 1972 by Schnabel, Lysmer, and Seed.

Ladd, C.C. and R. Foott. 1974. New Design Procedure for Stability of Soft Clays. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 100, No. 7, pp. 763-786.

Lambe and Whitman. 1969. *Soil Mechanics*. John Wiley & Sons, Inc.

Makdisi, F.I. and H.B. Seed. 1978. Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformation. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 104, No. GT7. July. pp. 849-868.

MegaTec Corporation. 1998. ProSheet Computer Program for Analyzing Sheet Pile Walls Using Blum Method. Version 1.2, Distributed by PROFIL ARBED & MegaTec, Programmed by Marco Mascarin and Dario Zogg.

Newmark, N.N. 1965. *Effects of Earthquakes on Dams and Embankments*. Fifth Rankine Lecture, Geotechnique, Volume 15, No. 2, pp. 41-87.

Office of Management and Budget (OMB). 2003. *Appendix C, Discount Rates for Cost-Effectiveness, Lease Purchase, and Related Analyses*. OMB Circular No. A-94. Available at http://www.whitehouse.gov/omb/circulars/a094/a94_appx-c.html. January.

Pacific Aerial Surveys. 1949, 1957. Aerial Photographs, Alameda Point, Alameda, California.

Ramanujam, N., L.L. Holish, and W.H. Chen. 1978. *Post-Earthquake Stability Analysis of Earth dams*. Earthquake Engineering and Soil Dynamics, Proceedings of the ASCE Geotechnical Engineering Division, Specialty Conference, Pasadena.

Robertson, P.K. and C.E. Wride. 1997. *Cyclic Liquefaction and its Evaluation Based on SPT and CPT*. Proc., NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Tech Rep. NCEER 97-0022, T.L. Youd and I.M. Idriss, eds., National Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo. 41-87.

Seed, R. B. and R. Bonaparte. 1992. *Seismic Analysis and Design of Lined Waste Fills: Current Practice*. In Proceedings of Stability and Performance of Slopes and Embankments - II, Vol. 2, Berkeley, California, ASCE Geotechnical Special Publication No. 31, pp. 1521-1545.

Tetra Tech EM Inc. (TtEMI). 1999. *Final OU-3 Remedial Investigation Report*. Alameda Point, Alameda, California. Volumes 1-3. Rancho Cordova, California.

________________. 2001. *Final OU-3 Remedial Investigation Addendum, Volume I, Data Gap Summary Report*. Alameda Point, Alameda, California.

________________. 2002a. *Revised Draft Feasibility Study Report Operable Unit 3, Site 1 - 1943-1956 Disposal Area*, Alameda Point, Alameda, California.

________________. 2002b. *Draft Final OU-3 Remedial Investigation Addendum, Volume II, Cumulative Human Health Risk Assessment*. Alameda Point, Alameda, California.

U.S. Environmental Protection Agency (EPA). 1996. *The Role of Cost in the Superfund Remedy Selection Process.* Publication 9200-3-23 FS, EPA 540/F-96018, PB96-963245. September.

Youd, T.L., C.M. Hansen, and S.F. Bartlett. 2002. Revised MLR Equations for Prediction of Lateral Spread Displacement. Paper accepted for publication in the *Journal of Geotechnical and Geoenvironmental Engineering, ASCE.*

APPENDIX A

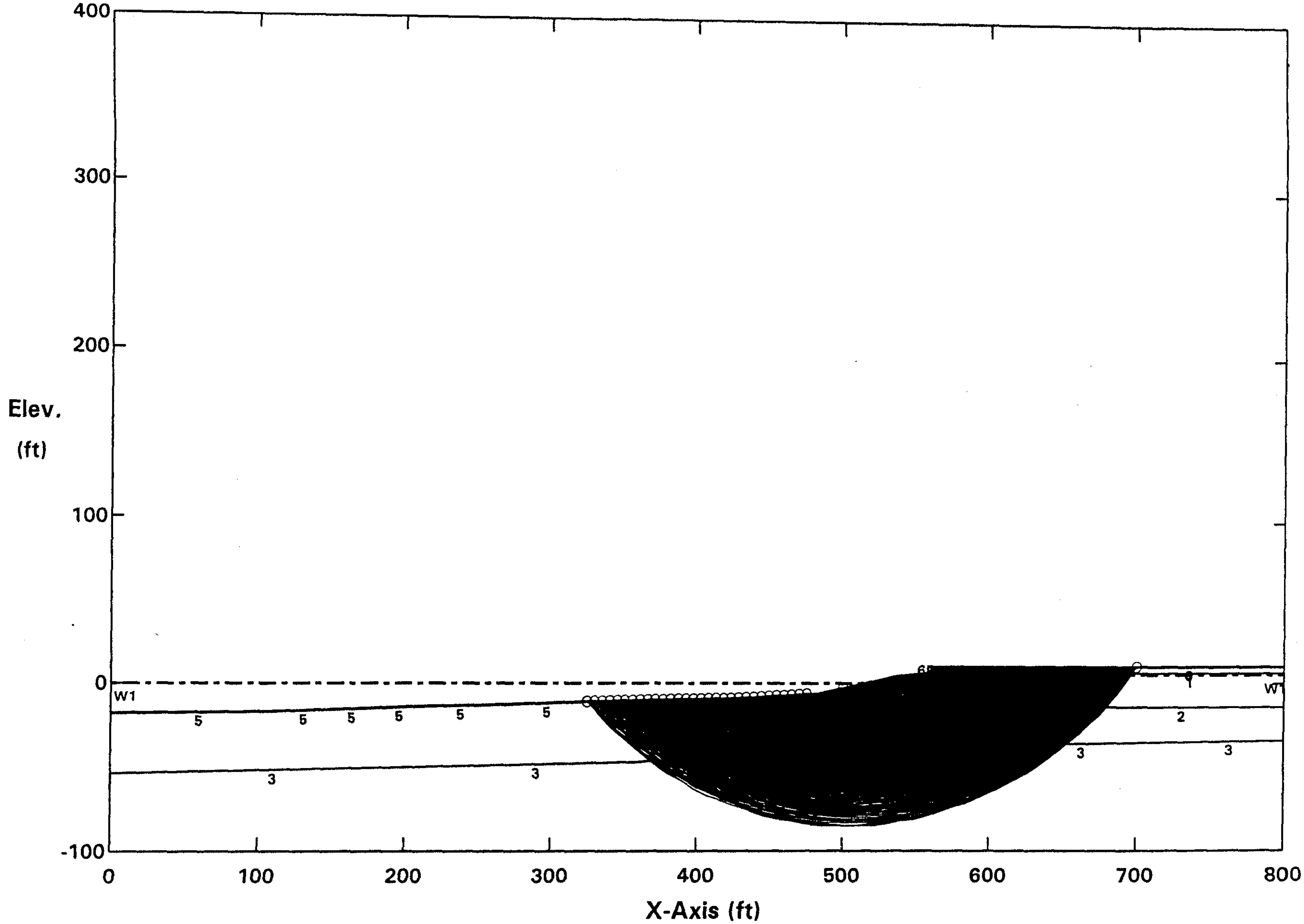
IMPLEMENTABILITY ANALYSIS

APPENDIX A1

ALTERNATIVE 1 – WICK DRAINS WITH SURCHARGE

A-NAS - Section D-D', Wick-Drain, 95 ft Static Bishop Circular Search

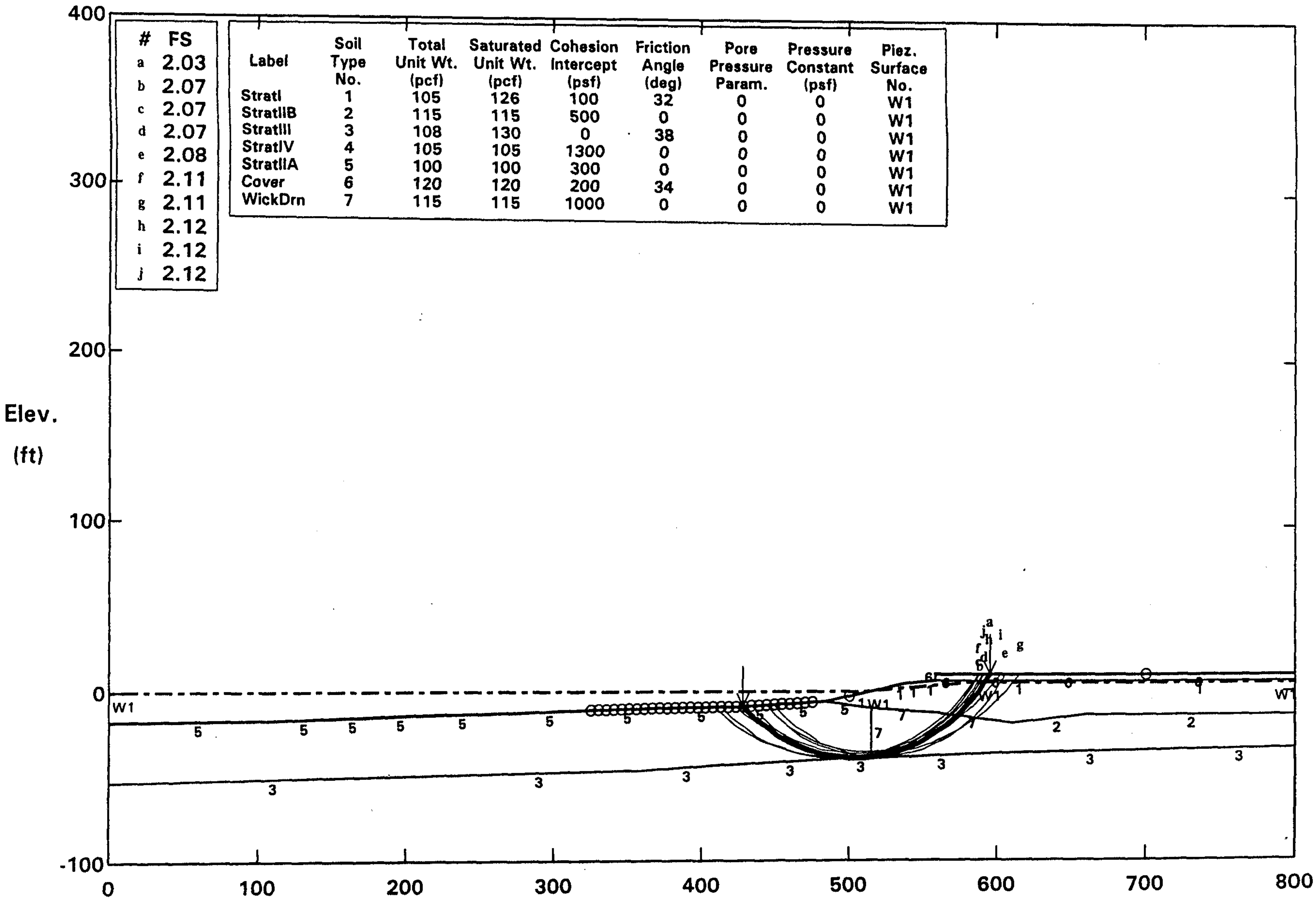
All surfaces evaluated. C:DWBS-1.PLT By: P.T. 06-29-02 6:15pm



Factors Of Safety Calculated By The Modified Bishop Method

A-NAS - Section D-D', Wick-Drain, 95 ft Static Bishop Circular Search

Ten Most Critical. C:DWBS-1.PLT By: P.T. 06-29-02 6:15pm



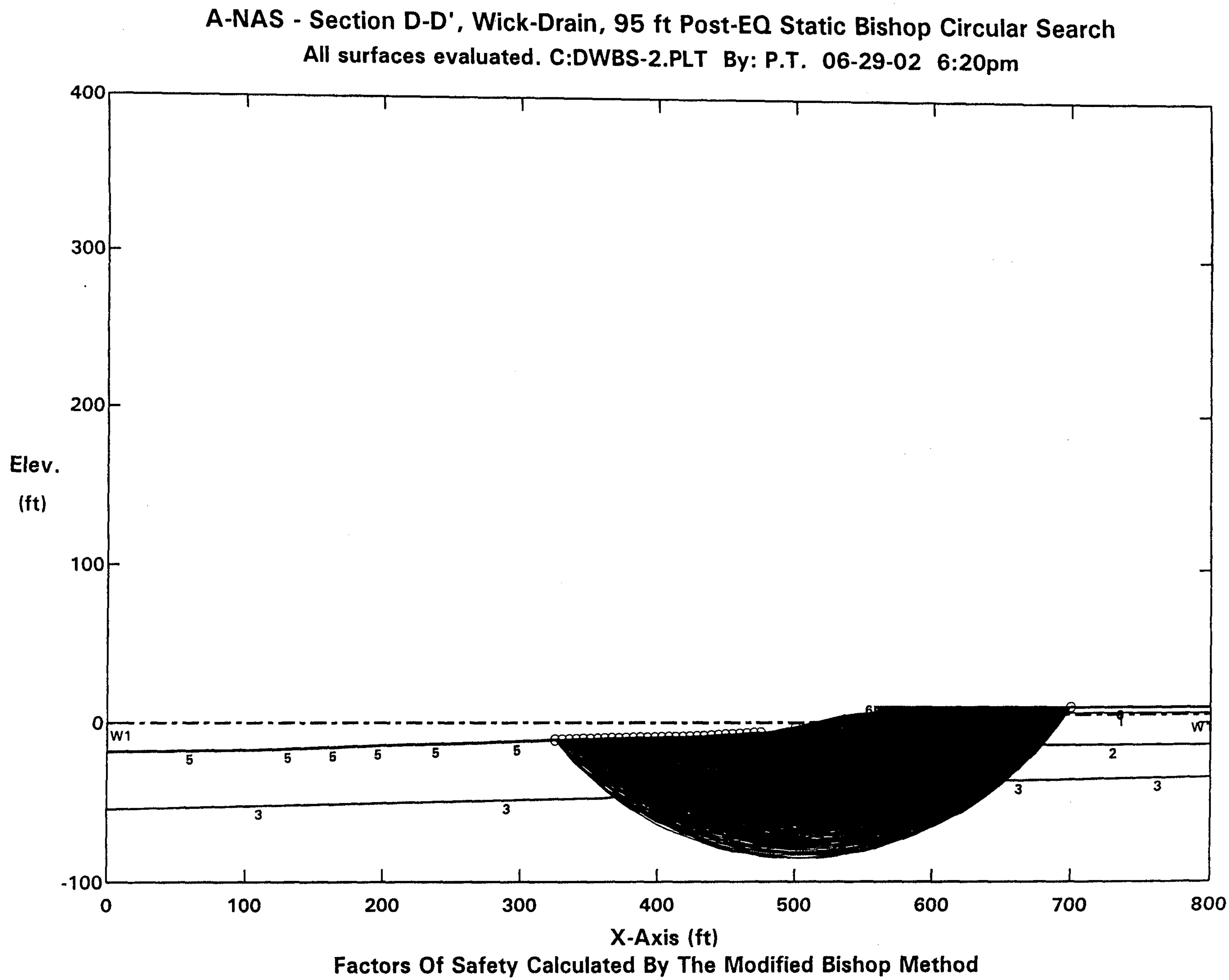
#	FS
a	2.03
b	2.07
c	2.07
d	2.07
e	2.08
f	2.11
g	2.11
h	2.12
i	2.12
j	2.12

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	100	32	0	0	W1
StratIIB	2	115	115	500	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1300	0	0	0	W1
StratIIA	5	100	100	300	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
WickDrn	7	115	115	1000	0	0	0	W1

PCSTABL5M FSmin = 2.03 X-Axis (ft)

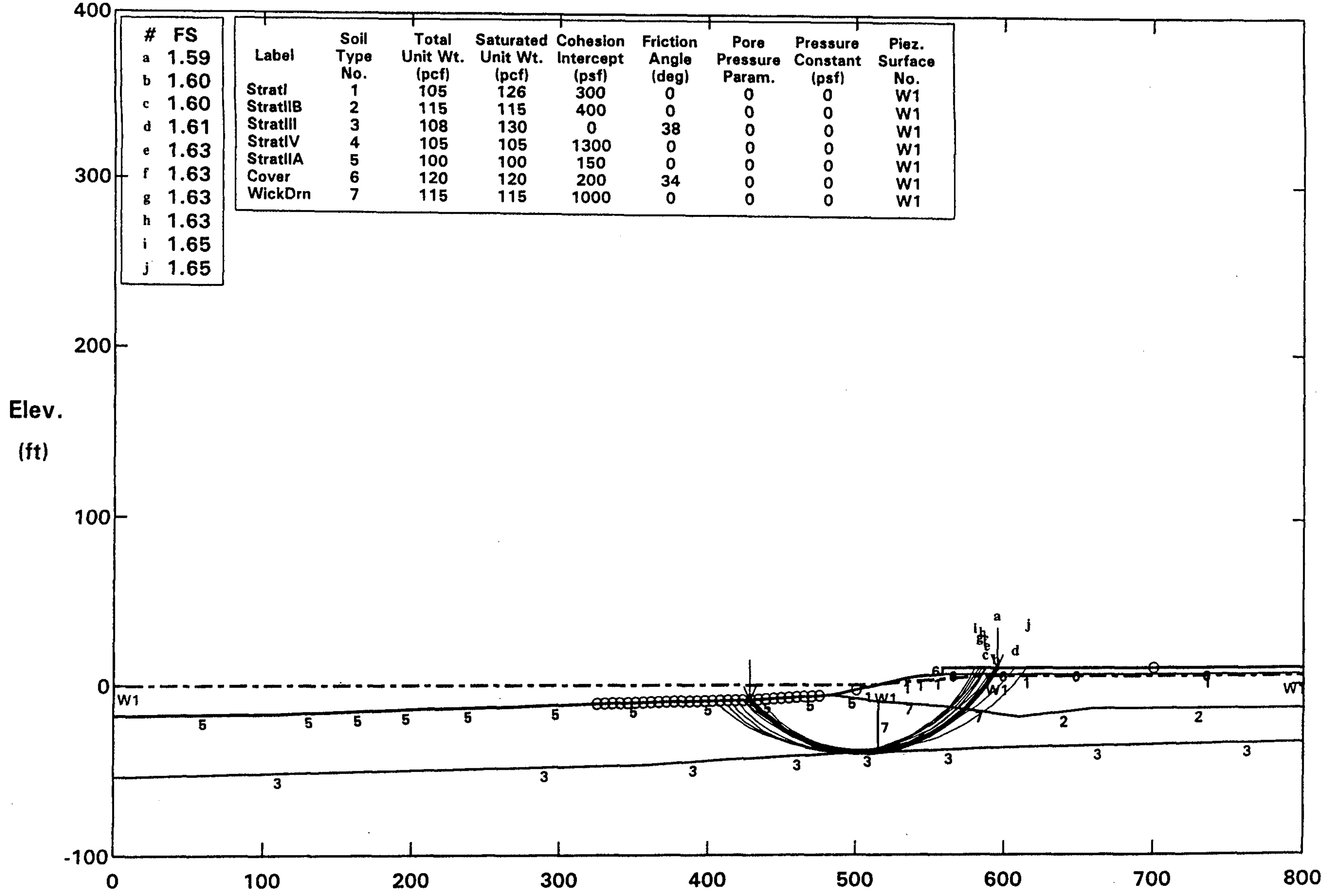
Factors Of Safety Calculated By The Modified Bishop Method

```
PROFIL        C:\GEO\STED\A-NAS\D-D\DRAIN\DWBS-1.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section D-D', Wick-Drain, 95 ft Static Bishop Circular Search
35 19
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 525. 102.1 1
525. 102.1 535. 104.1 1
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 560. 87. 7
560. 87. 610. 81.5 7
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   WickDrn
7
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1300. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
115. 115. 1000. 0. 0. 0. 1
WATER
1 62.4
4
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2-Bishop circular, search
30 100
325. 475. 500. 700. 0. 10. 0. 0.
```


A-NAS - Section D-D', Wick-Drain, 95 ft Post-EQ Static Bishop Circular Search
All surfaces evaluated. C:DWBS-2.PLT By: P.T. 06-29-02 6:20pm
Elev.
(ft)
400
300
200
100
0
-100
W1
5
3
6
2
0
100
200
300
400
500
600
700
800
X-Axis (ft)
Factors Of Safety Calculated By The Modified Bishop Method

A-NAS - Section D-D', Wick-Drain, 95 ft Post-EQ Static Bishop Circular Search

Ten Most Critical. C:DWBS-2.PLT By: P.T. 06-29-02 6:20pm



#	FS
a	1.59
b	1.60
c	1.60
d	1.61
e	1.63
f	1.63
g	1.63
h	1.63
i	1.65
j	1.65

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	300	0	0	0	W1
StratIIB	2	115	115	400	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1300	0	0	0	W1
StratIIA	5	100	100	150	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
WickDrn	7	115	115	1000	0	0	0	W1

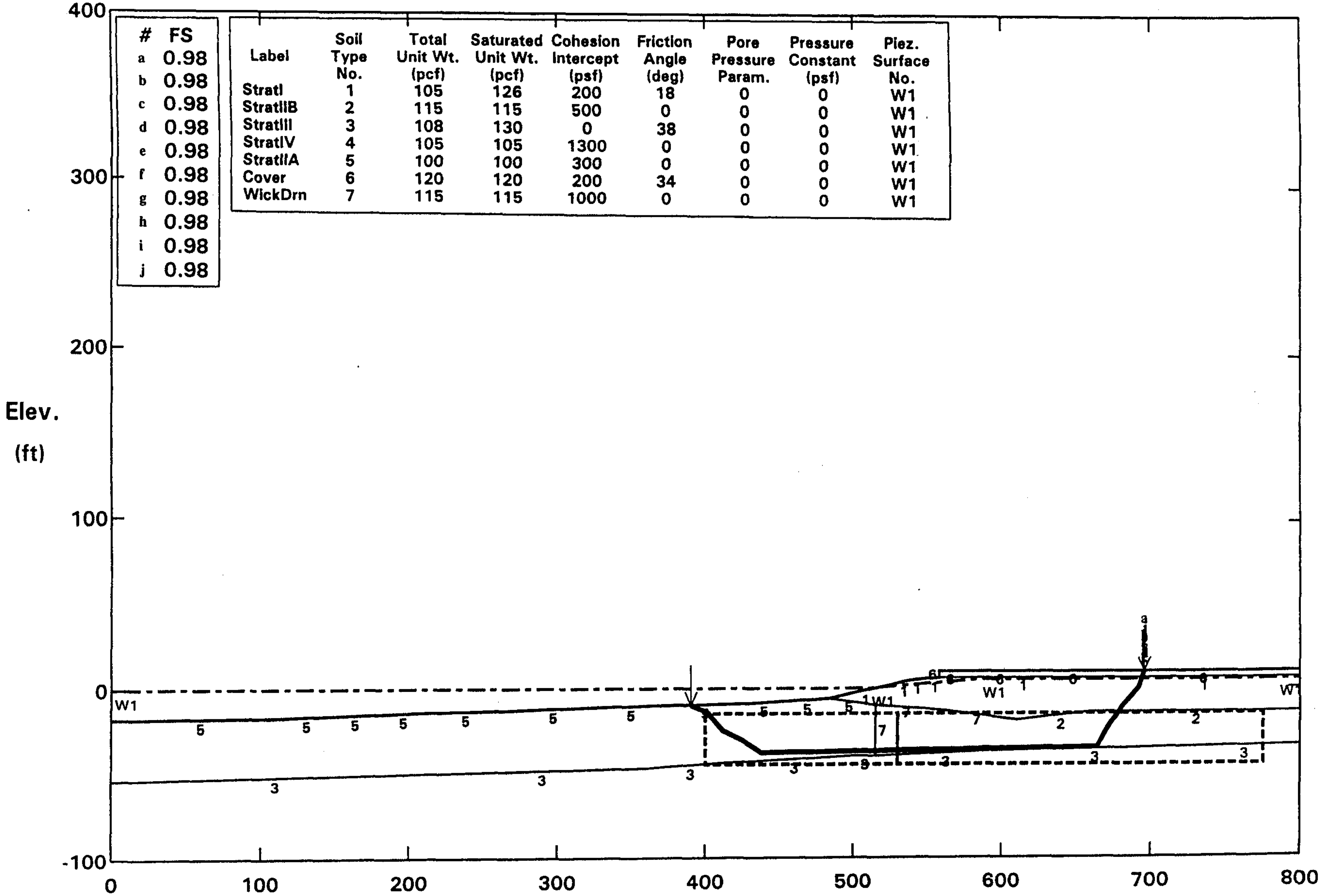
PCSTABL5M FSmin=1.59 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

```
PROFIL       C:\GEO\STED\A-NAS\D-D\DRAIN\DWBS-2.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section D-D', Wick-Drain, 95 ft Post-EQ Static Bishop Circular Search
35 19
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 525. 102.1 1
525. 102.1 535. 104.1 1
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 560. 87. 7
560. 87. 610. 81.5 7
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   WickDrn
7
105. 126. 300. 0. 0. 0. 1
115. 115. 400. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1300. 0. 0. 0. 1
100. 100. 150. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
115. 115. 1000. 0. 0. 0. 1
WATER
1 62.4
4
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2-Bishop circular, search
30 100
325. 475. 500. 700. 0. 10. 0. 0.
```

A-NAS - Section D-D', Wick-Drain, 95 ft Seismic,Block Failure Surface, Ky=0.11g

Ten Most Critical. C:DWBLD-1.PLT By: P.T. 06-29-02 6:09pm



#	FS
a	0.98
b	0.98
c	0.98
d	0.98
e	0.98
f	0.98
g	0.98
h	0.98
i	0.98
j	0.98

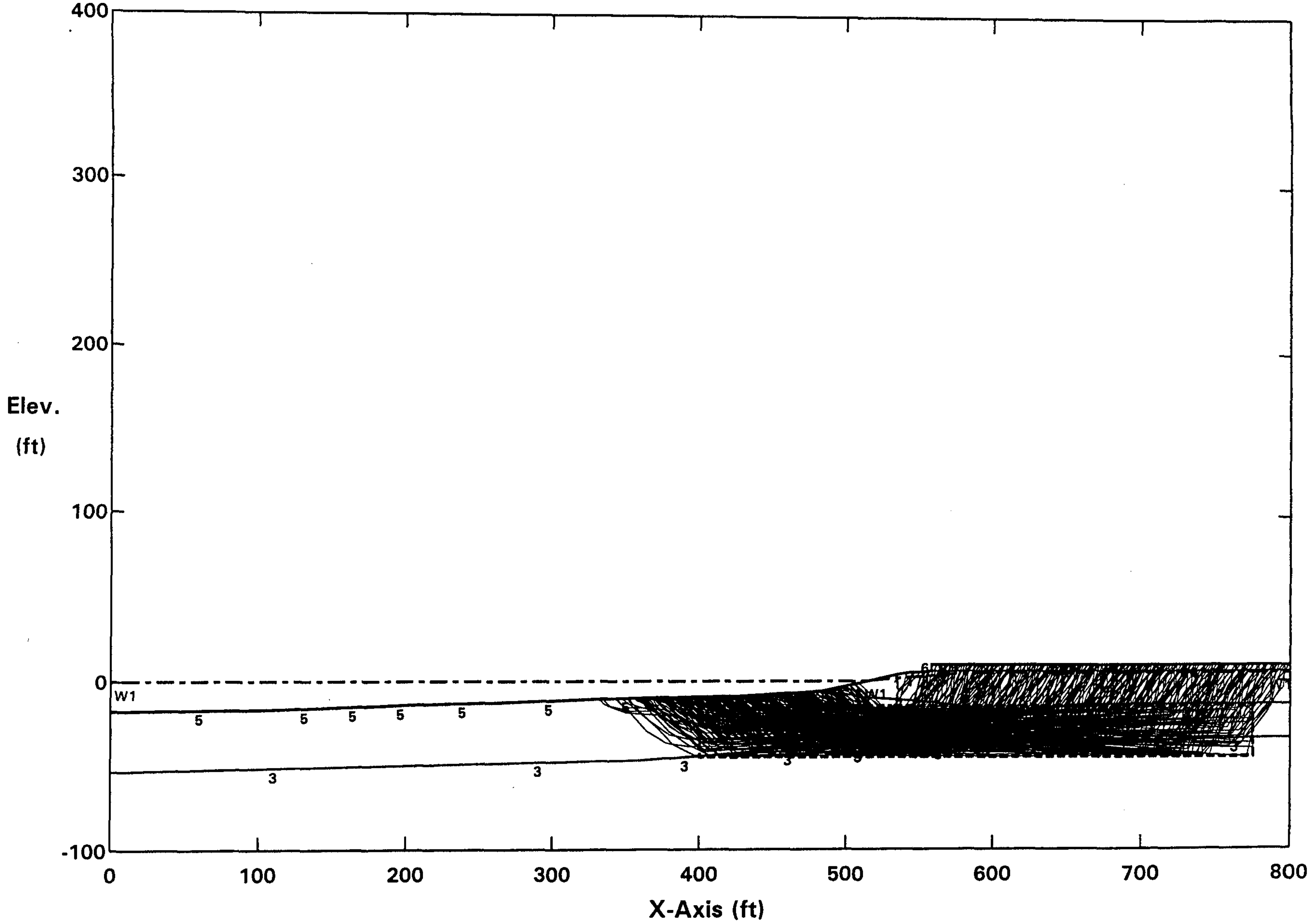
Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	200	18	0	0	W1
StratIIB	2	115	115	500	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1300	0	0	0	W1
StratIIA	5	100	100	300	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
WickDrn	7	115	115	1000	0	0	0	W1

PCSTABL5M FSmin=0.98 X-Axis (ft)

Factors Of Safety Calculated By The Modified Janbu Method

A-NAS - Section D-D', Wick-Drain, 95 ft Seismic,Block Failure Surface, Ky=0.11g

All surfaces evaluated. C:DWBLD-1.PLT By: P.T. 06-29-02 6:09pm



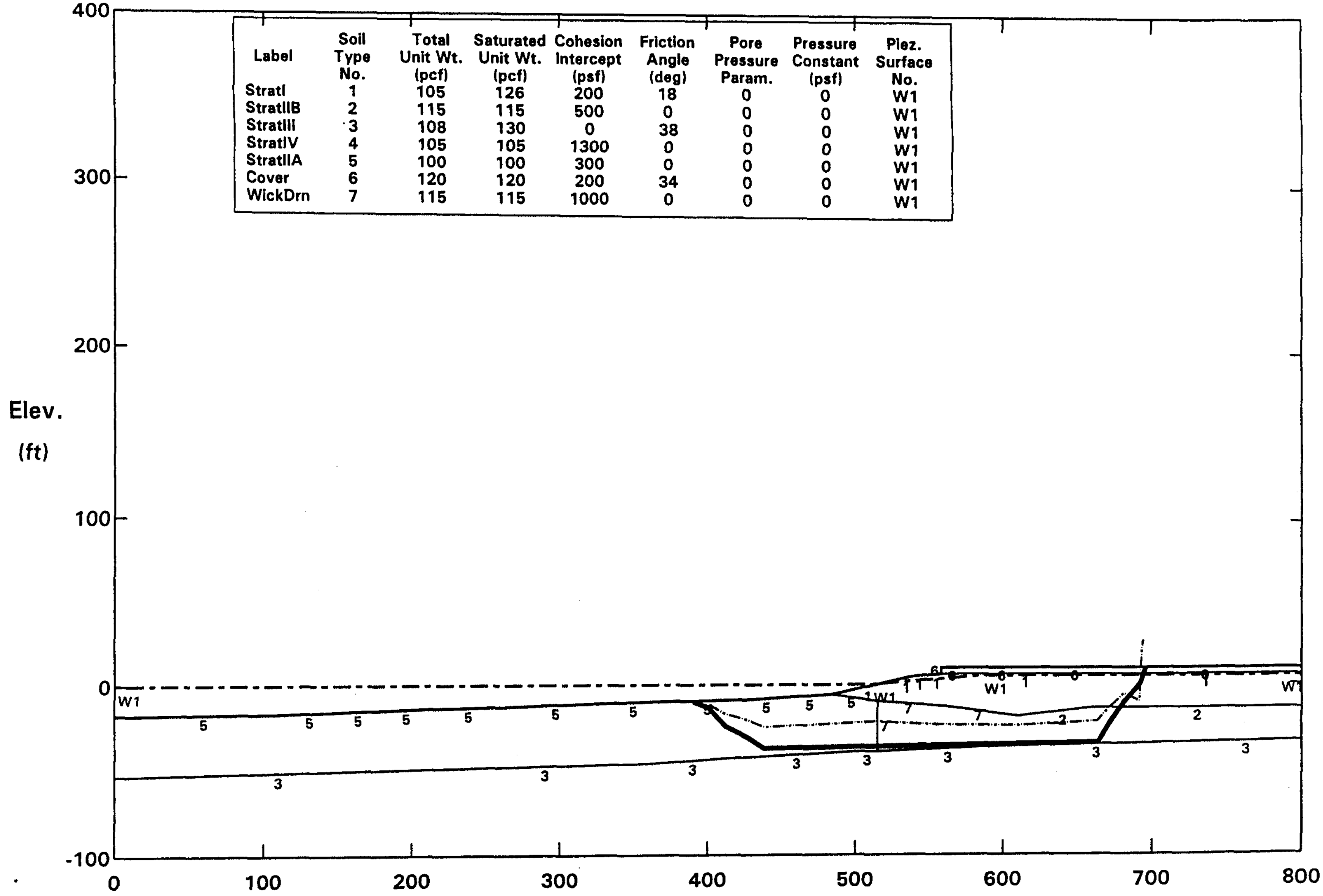
Factors Of Safety Calculated By The Modified Janbu Method

```
PROFIL       C:\GEO\STED\A-NAS\D-D\DRAIN\DWBLD-1.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section D-D', Wick-Drain, 95 ft Seismic,Block Failure Surface, Ky=0.11g
35 19
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 525. 102.1 1
525. 102.1 535. 104.1 1
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 560. 87. 7
560. 87. 610. 81.5 7
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   WickDrn
7
105. 126. 200. 18. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1300. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
115. 115. 1000. 0. 0. 0. 1
WATER
1 62.4
4
0. 100.
510. 100.
585. 105.
800. 105.
EQUAKE
0.11 0. 0.
```

```
BLOCK -Sliding block, search
0
4000 2 15.
400. 70. 529.9 70. 30.
530. 70. 775. 70. 30.
```

A-NAS - Section D-D', Wick-Drain, 95 ft Seismic,Block Failure Surface, Ky=0.11g
Surface #1-DWBLD-1.OUT. C:DWBLD-1S.PLT By: P.T. 06-29-02 6:11pm

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	200	18	0	0	W1
StratIIB	2	115	115	500	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1300	0	0	0	W1
StratIIA	5	100	100	300	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
WickDrn	7	115	115	1000	0	0	0	W1

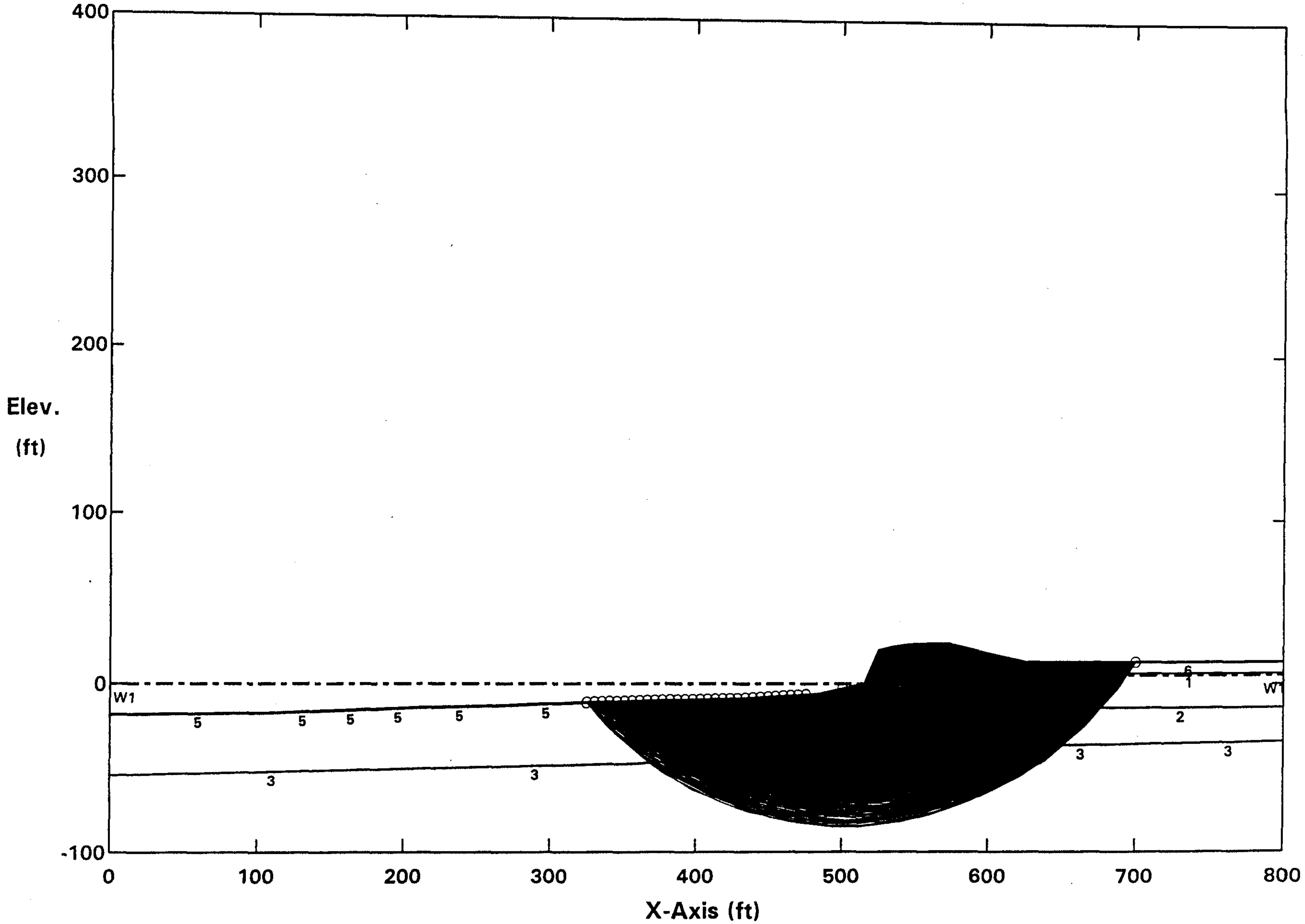


PCSTABL5M FS=1.00 Theta=1.53 X-Axis (ft)
Factors Of Safety Calculated By Spencer`s Method of Slices

```
PROFIL      C:\GEO\STED\A-NAS\D-D\DRAIN\DWBLD-1S.IN  PCSTABL Version 5M /O(0.
,-100. )
A-NAS - Section D-D', Wick-Drain, 95 ft Seismic,Block Failure Surface, Ky=0.11g
35 19
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 525. 102.1 1
525. 102.1 535. 104.1 1
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 560. 87. 7
560. 87. 610. 81.5 7
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   WickDrn
7
105. 126. 200. 18. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1300. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
115. 115. 1000. 0. 0. 0. 1
WATER
1 62.4
4
0. 100.
510. 100.
585. 105.
800. 105.
EQUAKE
0.11 0. 0.
```

```
SPENCR
10.
SURFAC #1-DWBLD-1.OUT
10
389.83 90.41
401.07 85.78
411.69 75.19
425.55 69.45
438.4 61.71
663.55 64.8
671.99 77.2
680.99 89.2
691.52 99.88
695.81 110.
EXECUT
```

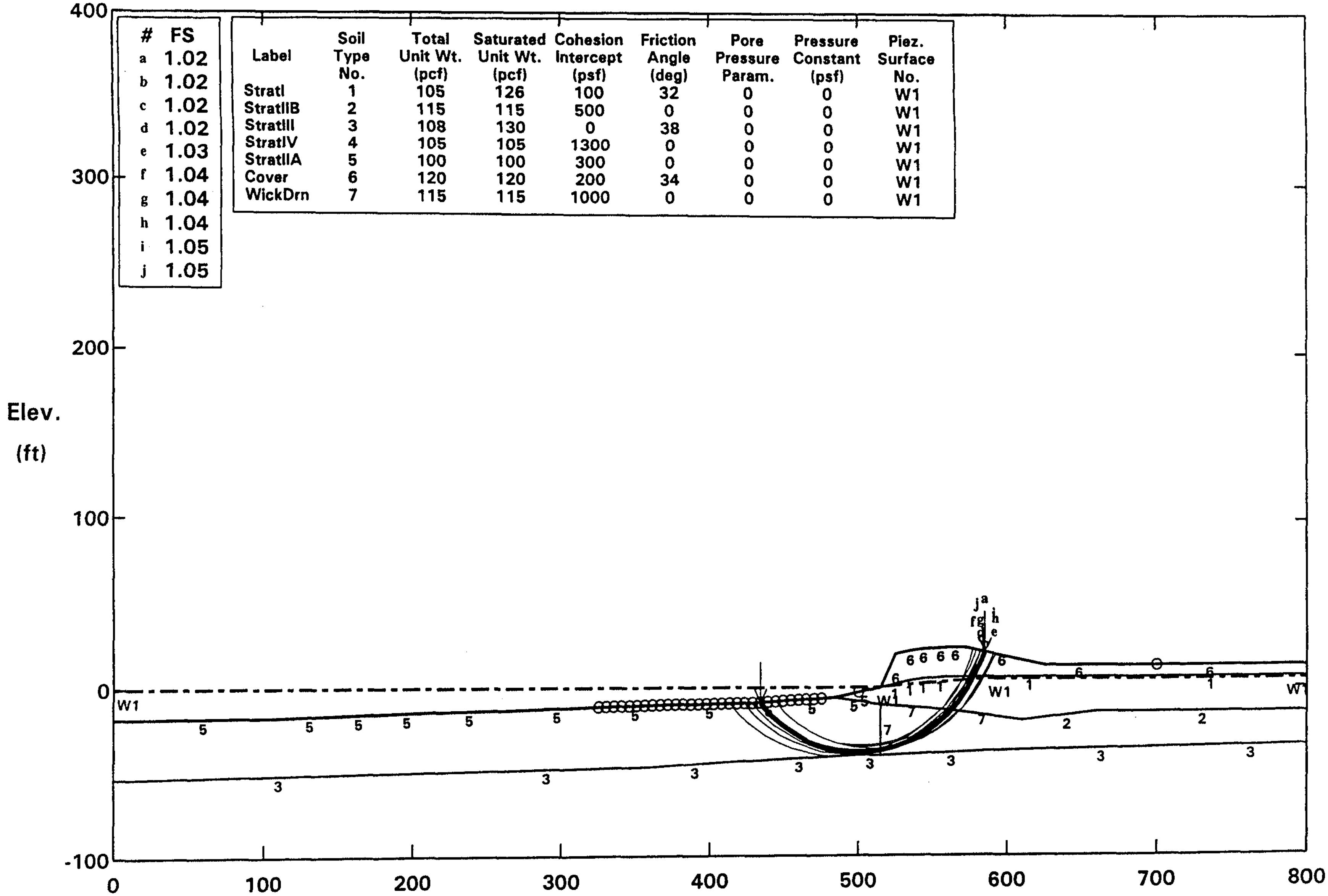
A-NAS, Section D-D'-Wick Drain Surcharge = 18 ft Soil Layer, Static Bishop Circular
All surfaces evaluated. C:DWBS.PLT By: P.T. 06-29-02 6:33pm



Factors Of Safety Calculated By The Modified Bishop Method

A-NAS, Section D-D'-Wick Drain Surcharge = 18 ft Soil Layer, Static Bishop Circular

Ten Most Critical. C:DWBS.PLT By: P.T. 06-29-02 6:33pm



#	FS
a	1.02
b	1.02
c	1.02
d	1.02
e	1.03
f	1.04
g	1.04
h	1.04
i	1.05
j	1.05

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	100	32	0	0	W1
StratIIB	2	115	115	500	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1300	0	0	0	W1
StratIIA	5	100	100	300	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
WickDrn	7	115	115	1000	0	0	0	W1

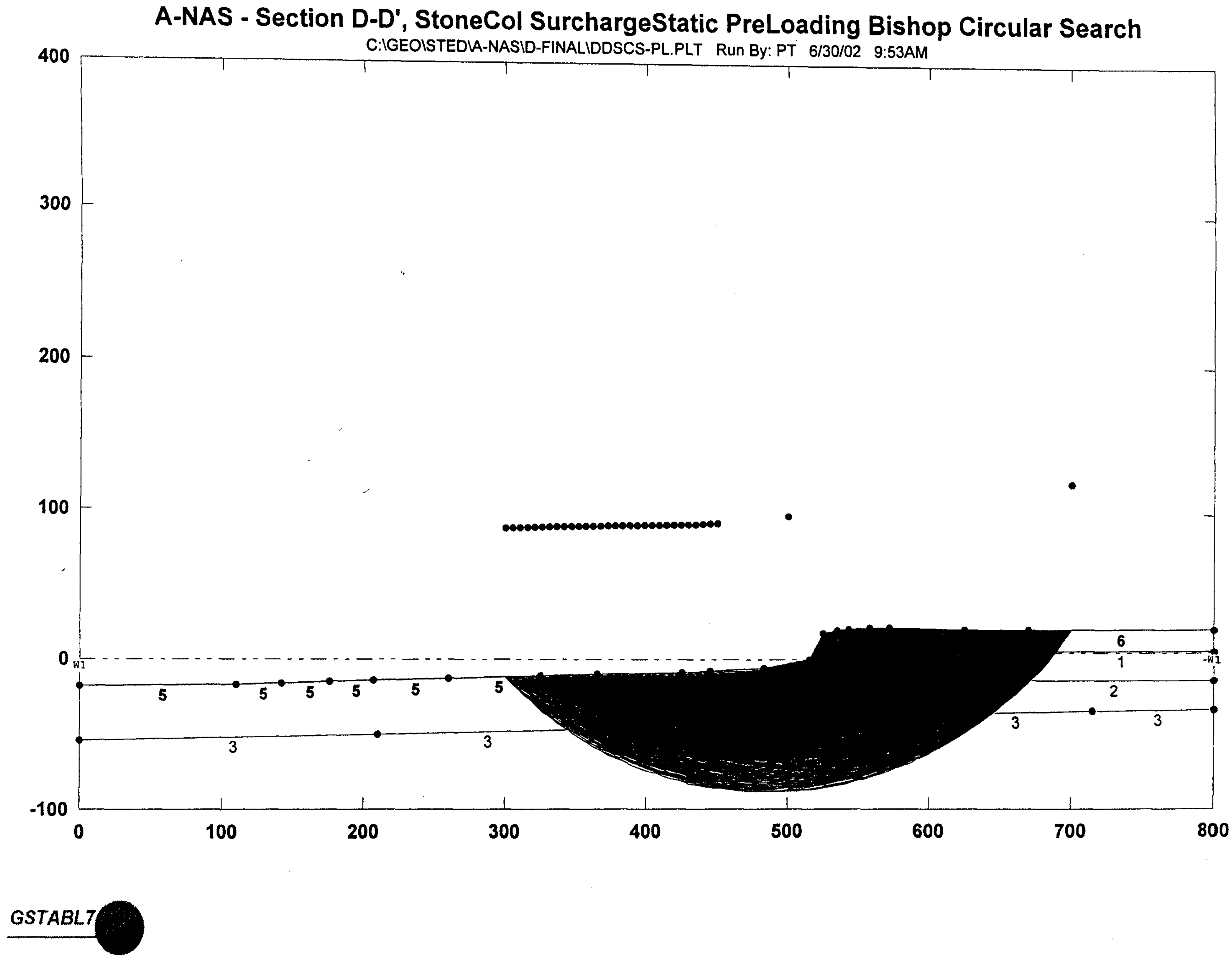
PCSTABL5M FSmin = 1.02 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

```
PROFIL       C:\GEO\STED\A-NAS\D-D\DRAIN\DWBS.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS, Section D-D'-Wick Drain Surcharge=18 ft Soil Layer,Static Bishop
Circular
39 19
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515. 100.2 5
515. 100.2 525. 120.1 6
525. 120.1 535. 122.1 6
535. 122.1 543. 123.1 6
543. 123.1 558. 124.1 6
558. 124.1 572. 124.1 6
572. 124.1 625. 113.1 6
625. 113.1 670. 113.1 6
670. 113.1 800. 112.6 6
515. 100.2 525. 102.1 1
525. 102.1 535. 104.1 1
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 560. 87. 7
560. 87. 610. 81.5 7
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   WickDrn
7
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1300. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
115. 115. 1000. 0. 0. 0. 1
WATER
1 62.4
4
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2-Bishop circular, search
30 100
325. 475. 500. 700. 0. 10. 0. 0.
```

APPENDIX A2

ALTERNATIVE 2 – STONE COLUMNS WITH SURCHARGE



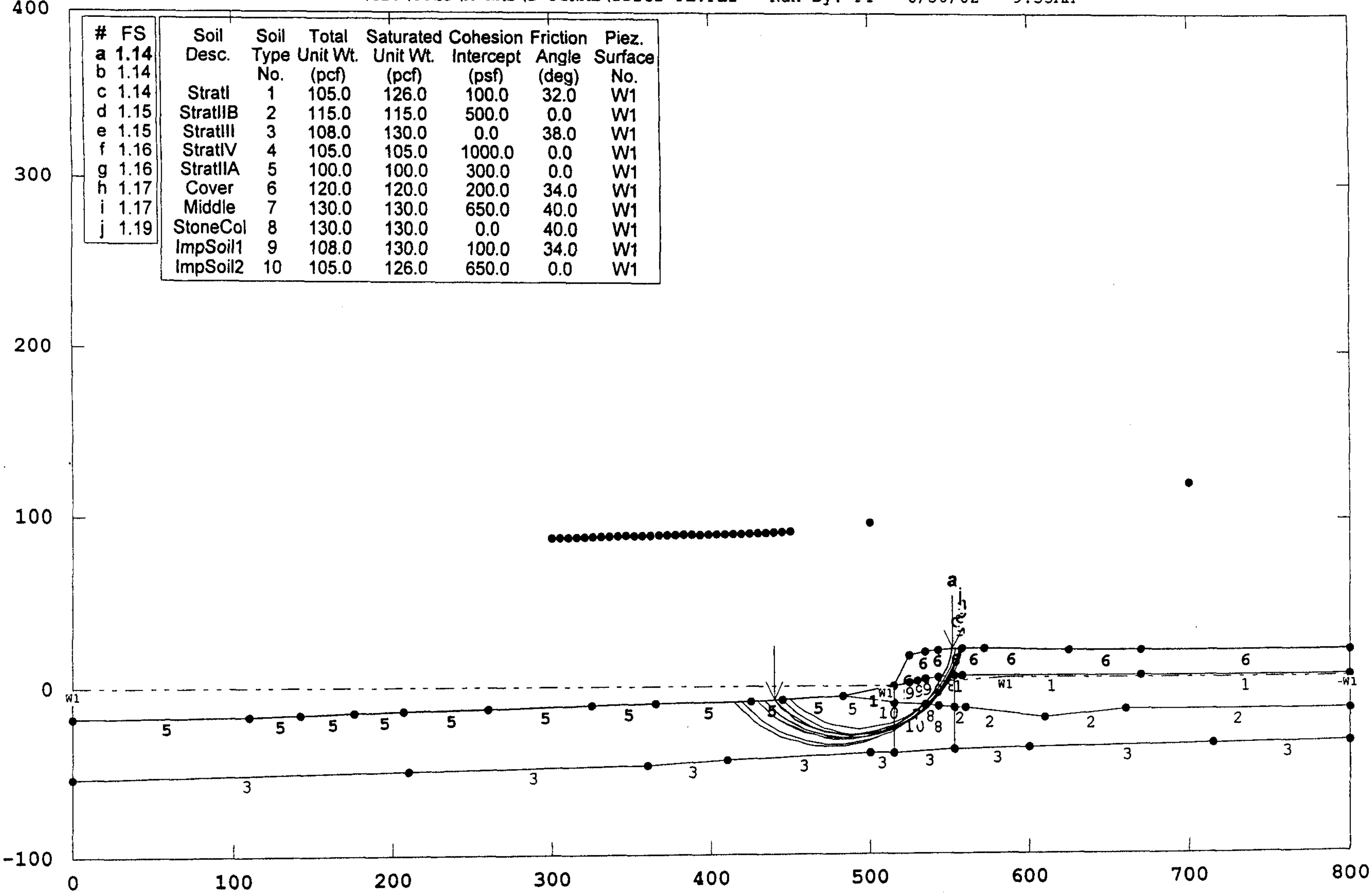
A-NAS - Section D-D', StoneCol SurchargeStatic PreLoading Bishop Circular Search
C:\GEO\STED\A-NAS\D-FINAL\DDSCS-PL.PLT Run By: PT 6/30/02 9:53AM
400
300
200
100
0
-100
0
100
200
300
400
500
600
700
800
W1
5
3
6
1
2
GSTABL7

A-NAS - Section D-D', StoneCol SurchargeStatic PreLoading Bishop Circular Search

C:\GEO\STED\A-NAS\D-FINAL\DDSCS-PL.PL2 Run By: PT 6/30/02 9:53AM

#	FS
a	**1.14**
b	1.14
c	1.14
d	1.15
e	1.15
f	1.16
g	1.16
h	1.17
i	1.17
j	1.19

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	100.0	32.0	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1000.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Middle	7	130.0	130.0	650.0	40.0	W1
StoneCol	8	130.0	130.0	0.0	40.0	W1
ImpSoil1	9	108.0	130.0	100.0	34.0	W1
ImpSoil2	10	105.0	126.0	650.0	0.0	W1



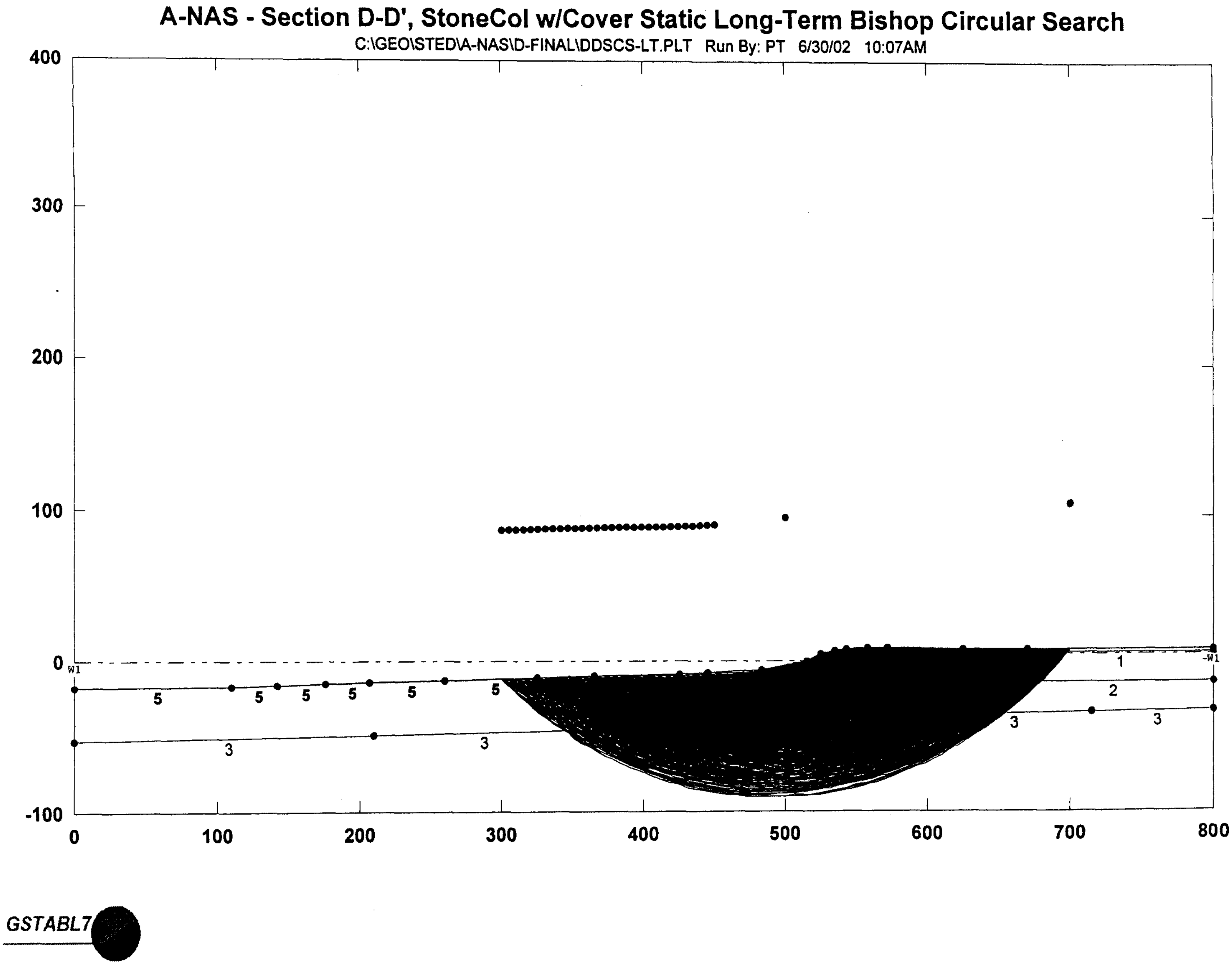
GSTABL7 v.2 FSmin=1.14

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

```
PROFIL  C:\geo\sted\a-nas\d-final\ddscs-pl.in Version G7v.2 [GSTABL72.EXE] /O(0,
-100)
e
A-NAS - Section D-D', StoneCol SurchargeStatic PreLoading Bishop Circular Search
48 19
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 525. 118.1 6
525. 118.1 535. 120.1 6
535. 120.1 543. 121.1 6
543. 121.1 558. 122.1 6
558. 122.1 572. 122.1 6
572. 122.1 625. 121.1 6
625. 121.1 670. 121.1 6
670. 121.1 800. 120.6 6
515.1 100.2 525. 102.1 9
525. 102.1 530. 103.1 9
530. 103.1 535. 104.1 9
535. 104.1 543. 105.1 8
543. 105.1 553. 105.77 8
553. 105.77 558. 106.1 1
515. 90. 515.1 100.2 9
553. 105.77 553.1 87.4 8
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 10
515. 90. 535. 88.67 10
535. 88.67 543. 88.11 7
543. 88.11 553.1 87.4 8
553.1 87.4 560. 87. 2
553.1 87.4 553.2 61.6 8
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 553.2 61.6 3
553.2 61.6 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
```

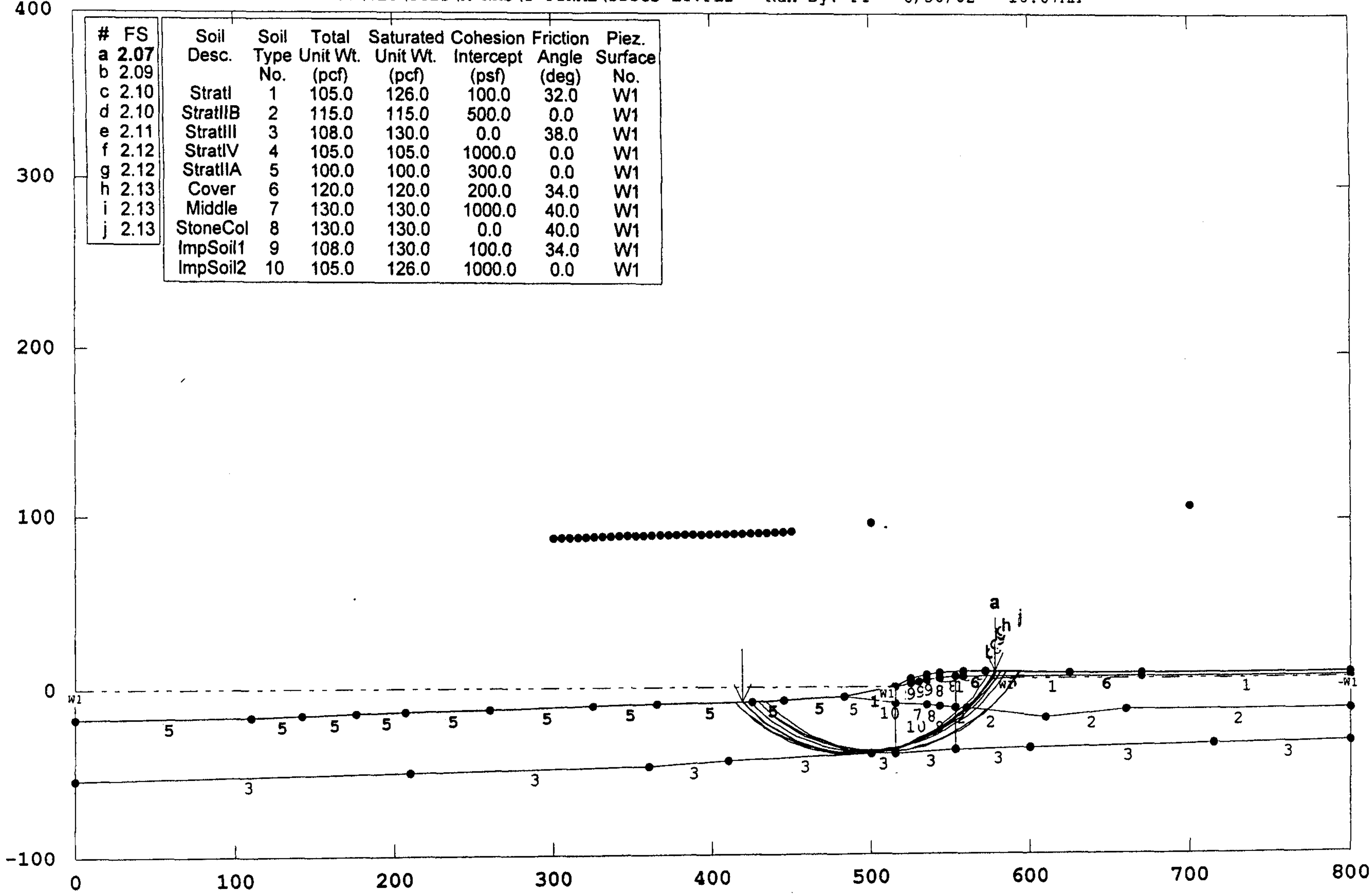
```
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 650. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 650. 0. 0. 0. 1
WATER
1 62.4
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2-Bishop circular, search
30 100
300. 450. 500. 700.
0. 6. 0. 0.
```



A-NAS - Section D-D', StoneCol w/Cover Static Long-Term Bishop Circular Search
C:\GEO\STED\A-NAS\D-FINAL\DDSCS-LT.PLT Run By: PT 6/30/02 10:07AM
400
300
200
100
0
-100
0
100
200
300
400
500
600
700
800
W1
5
3
1
2
GSTABL7

A-NAS - Section D-D', StoneCol w/Cover Static Long-Term Bishop Circular Search

C:\GEO\STED\A-NAS\D-FINAL\DDSCS-LT.PL2 Run By: PT 6/30/02 10:07AM



#	FS
a	2.07
b	2.09
c	2.10
d	2.10
e	2.11
f	2.12
g	2.12
h	2.13
i	2.13
j	2.13

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	100.0	32.0	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1000.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Middle	7	130.0	130.0	1000.0	40.0	W1
StoneCol	8	130.0	130.0	0.0	40.0	W1
ImpSoil1	9	108.0	130.0	100.0	34.0	W1
ImpSoil2	10	105.0	126.0	1000.0	0.0	W1

GSTABL7 v.2 FSmin=2.07

Safety Factors Are Calculated By The Modified Bishop Method

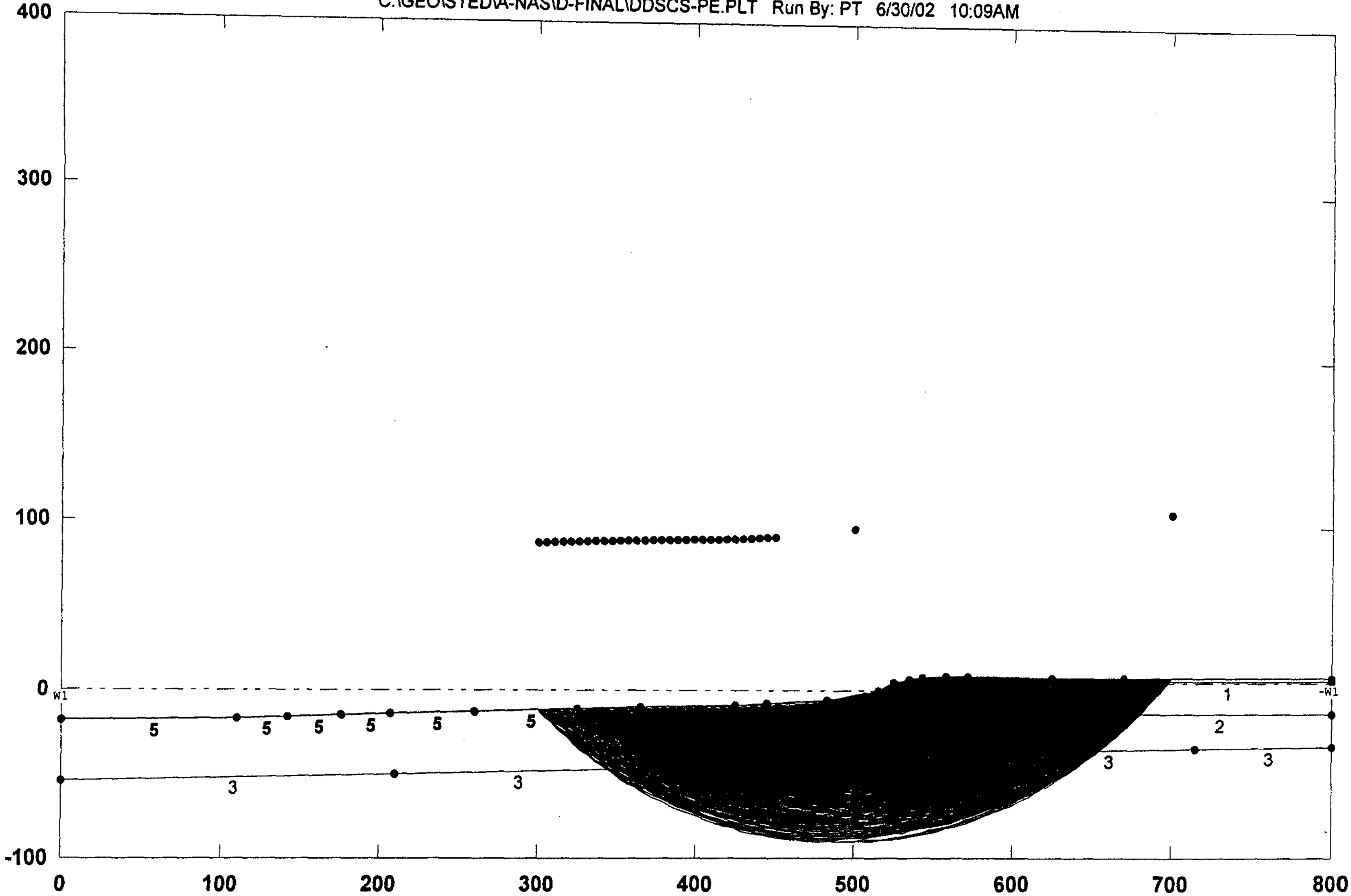
GSTABL7

```
PROFIL  c:\geo\sted\a-nas\d-final\ddscs-lt.in Version G7v.2 [GSTABL72.EXE] /O(0,
-100)
e
A-NAS - Section D-D', StoneCol w/Cover  Static Long-Term Bishop Circular Search
48 19
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 525. 105.1 6
525. 105.1 535. 107.1 6
535. 107.1 543. 108.1 6
543. 108.1 558. 109.1 6
558. 109.1 572. 109.1 6
572. 109.1 625. 108.1 6
625. 108.1 670. 108.1 6
670. 108.1 800. 107.6 6
515.1 100.2 525. 102.1 9
525. 102.1 530. 103.1 9
530. 103.1 535. 104.1 9
535. 104.1 543. 105.1 8
543. 105.1 553. 105.77 8
553. 105.77 558. 106.1 1
515. 90. 515.1 100.2 9
553. 105.77 553.1 87.4 8
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 10
515. 90. 535. 88.67 10
535. 88.67 543. 88.11 7
543. 88.11 553.1 87.4 8
553.1 87.4 560. 87. 2
553.1 87.4 553.2 61.6 8
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 553.2 61.6 3
553.2 61.6 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
```

```
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
WATER
1 62.4
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2-Bishop circular, search
30 100
300. 450. 500. 700.
0. 6. 0. 0.
```

A-NAS - Section D-D', StoneCol w/Cover Static Post-EQ Bishop Circular Search

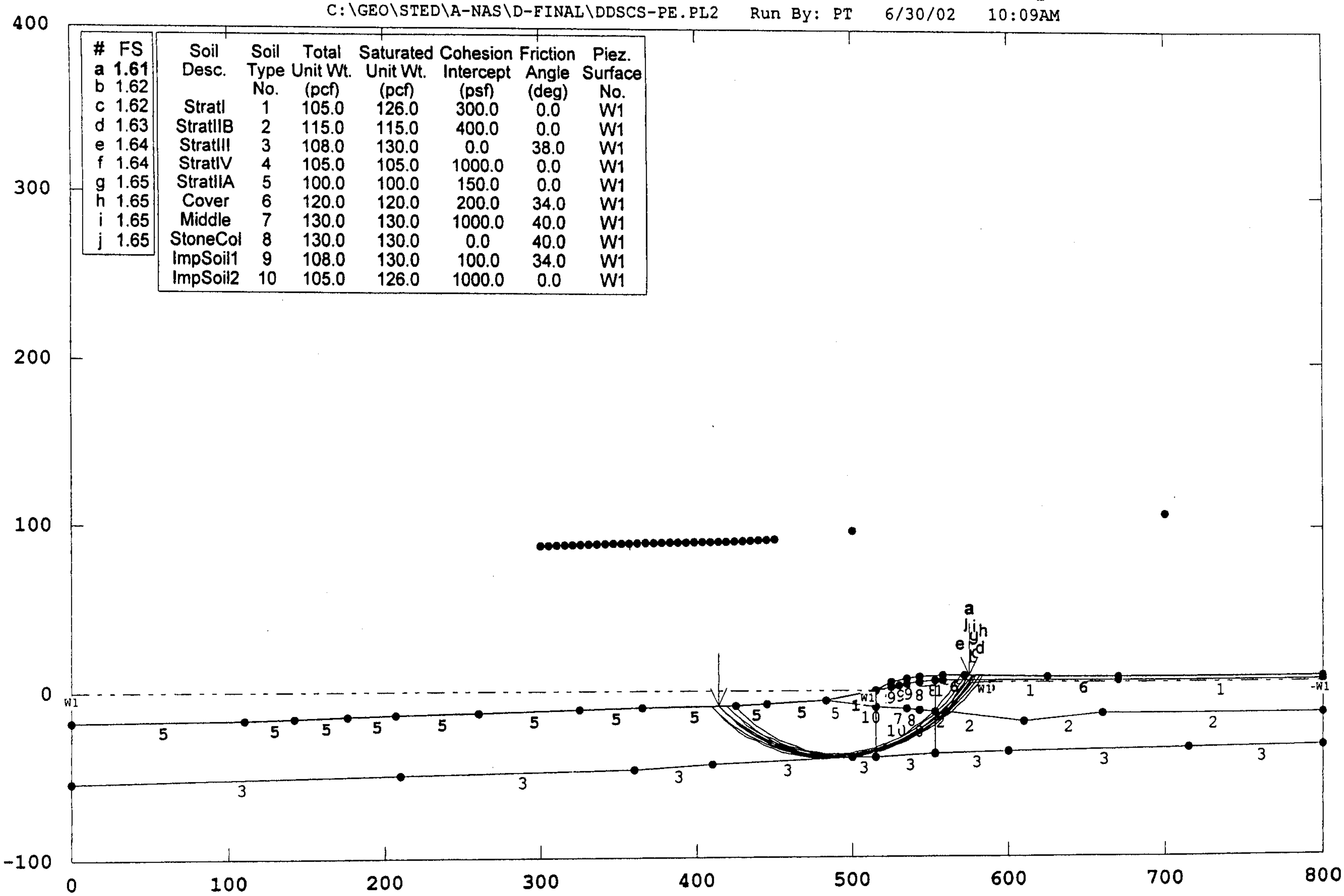
C:\GEO\STED\A-NAS\D-FINAL\DDSCS-PE.PLT Run By: PT 6/30/02 10:09AM



GSTABL7

A-NAS - Section D-D', StoneCol w/Cover Static Post-EQ Bishop Circular Search

C:\GEO\STED\A-NAS\D-FINAL\DDSCS-PE.PL2   Run By: PT   6/30/02   10:09AM



#	FS
a	1.61
b	1.62
c	1.62
d	1.63
e	1.64
f	1.64
g	1.65
h	1.65
i	1.65
j	1.65

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	300.0	0.0	W1
StratIIB	2	115.0	115.0	400.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1000.0	0.0	W1
StratIIA	5	100.0	100.0	150.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Middle	7	130.0	130.0	1000.0	40.0	W1
StoneCol	8	130.0	130.0	0.0	40.0	W1
ImpSoil1	9	108.0	130.0	100.0	34.0	W1
ImpSoil2	10	105.0	126.0	1000.0	0.0	W1

GSTABL7 v.2 FSmin=1.61

Safety Factors Are Calculated By The Modified Bishop Method

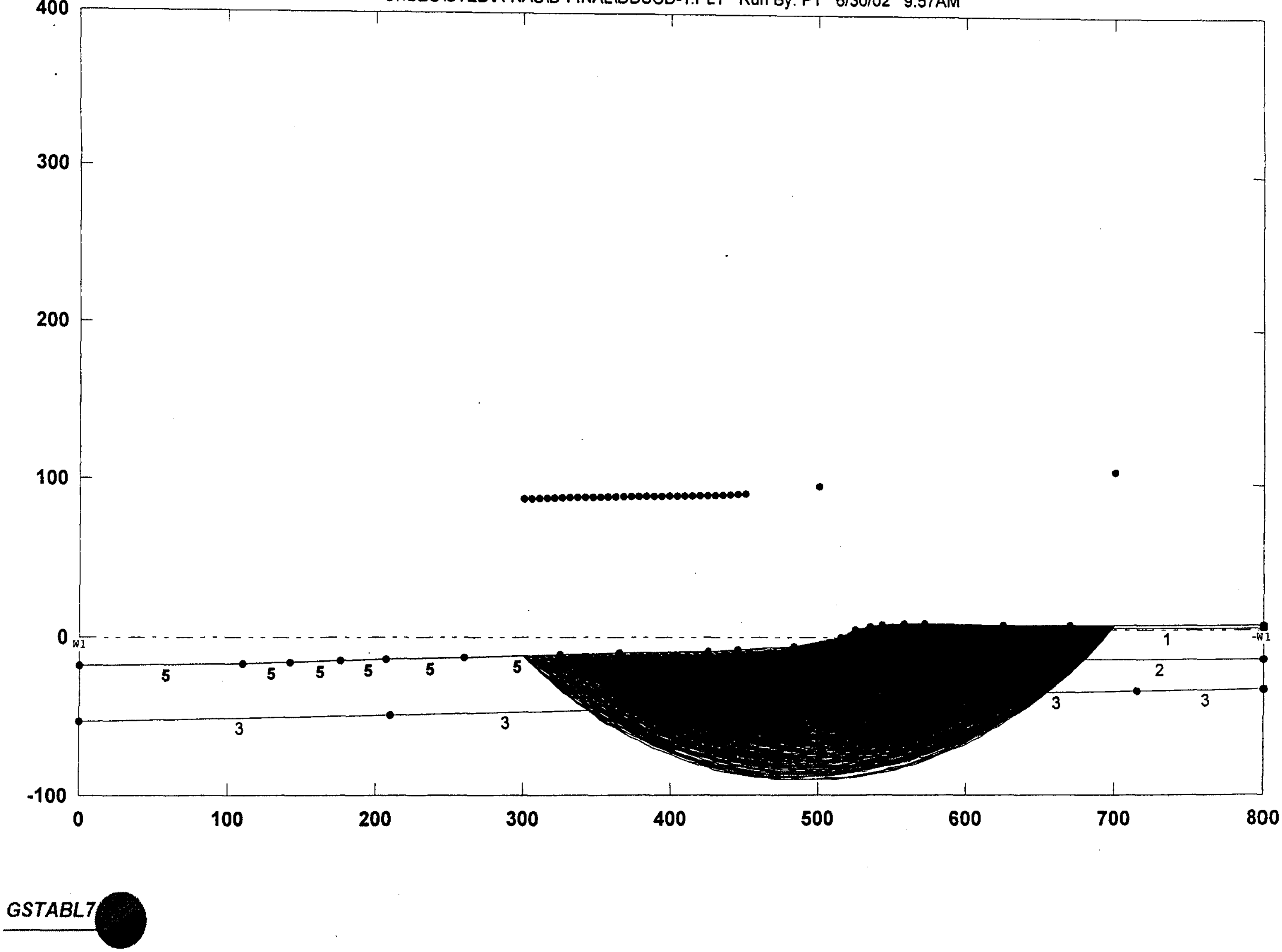
GSTABL7

```
PROFIL  c:\geo\sted\a-nas\d-final\ddscs-pe.in Version G7v.2 [GSTABL72.EXE] /O(0,
-100)
e
A-NAS - Section D-D', StoneCol w/Cover  Static Post-EQ Bishop Circular Search
48 19
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 525. 105.1 6
525. 105.1 535. 107.1 6
535. 107.1 543. 108.1 6
543. 108.1 558. 109.1 6
558. 109.1 572. 109.1 6
572. 109.1 625. 108.1 6
625. 108.1 670. 108.1 6
670. 108.1 800. 107.6 6
515.1 100.2 525. 102.1 9
525. 102.1 530. 103.1 9
530. 103.1 535. 104.1 9
535. 104.1 543. 105.1 8
543. 105.1 553. 105.77 8
553. 105.77 558. 106.1 1
515. 90. 515.1 100.2 9
553. 105.77 553.1 87.4 8
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 10
515. 90. 535. 88.67 10
535. 88.67 543. 88.11 7
543. 88.11 553.1 87.4 8
553.1 87.4 560. 87. 2
553.1 87.4 553.2 61.6 8
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 553.2 61.6 3
553.2 61.6 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
```

```
105. 126. 300. 0. 0. 0. 1
115. 115. 400. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 150. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
WATER
1 62.4
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2-Bishop circular, search
30 100
300. 450. 500. 700.
0. 6. 0. 0.
```

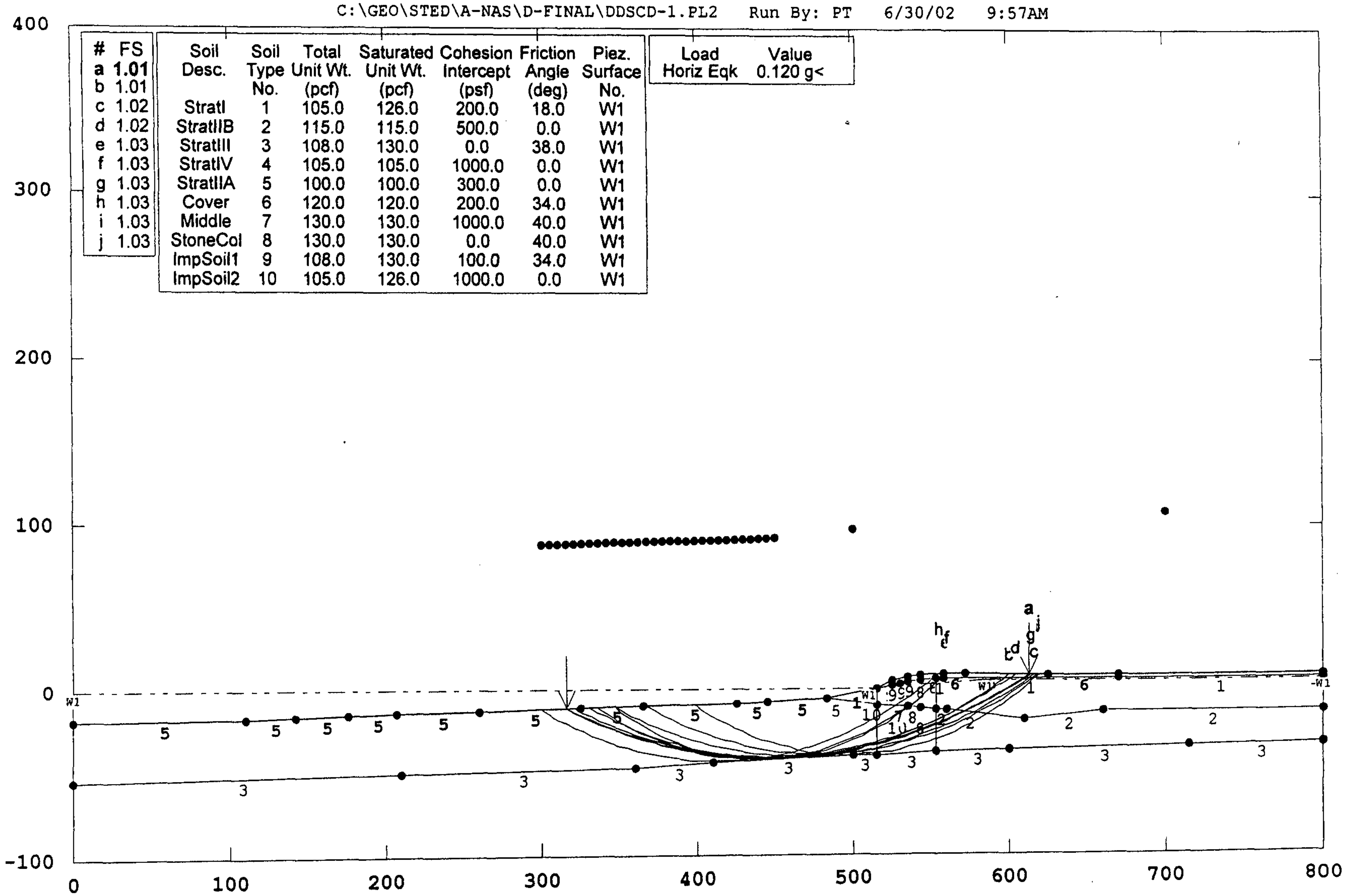
A-NAS - Section D-D', StoneCol w/Cover Dynamic Bishop Circular Search

C:\GEO\STED\A-NAS\D-FINAL\DDSCD-1.PLT Run By: PT 6/30/02 9:57AM



A-NAS - Section D-D', StoneCol w/Cover Dynamic Bishop Circular Search

C:\GEO\STED\A-NAS\D-FINAL\DDSCD-1.PL2   Run By: PT   6/30/02   9:57AM



#	FS
a	**1.01**
b	1.01
c	1.02
d	1.02
e	1.03
f	1.03
g	1.03
h	1.03
i	1.03
j	1.03

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	200.0	18.0	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1000.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Middle	7	130.0	130.0	1000.0	40.0	W1
StoneCol	8	130.0	130.0	0.0	40.0	W1
ImpSoil1	9	108.0	130.0	100.0	34.0	W1
ImpSoil2	10	105.0	126.0	1000.0	0.0	W1

Load	Value
Horiz Eqk	0.120 g<

GSTABL7 v.2 FSmin=1.01

Safety Factors Are Calculated By The Modified Bishop Method

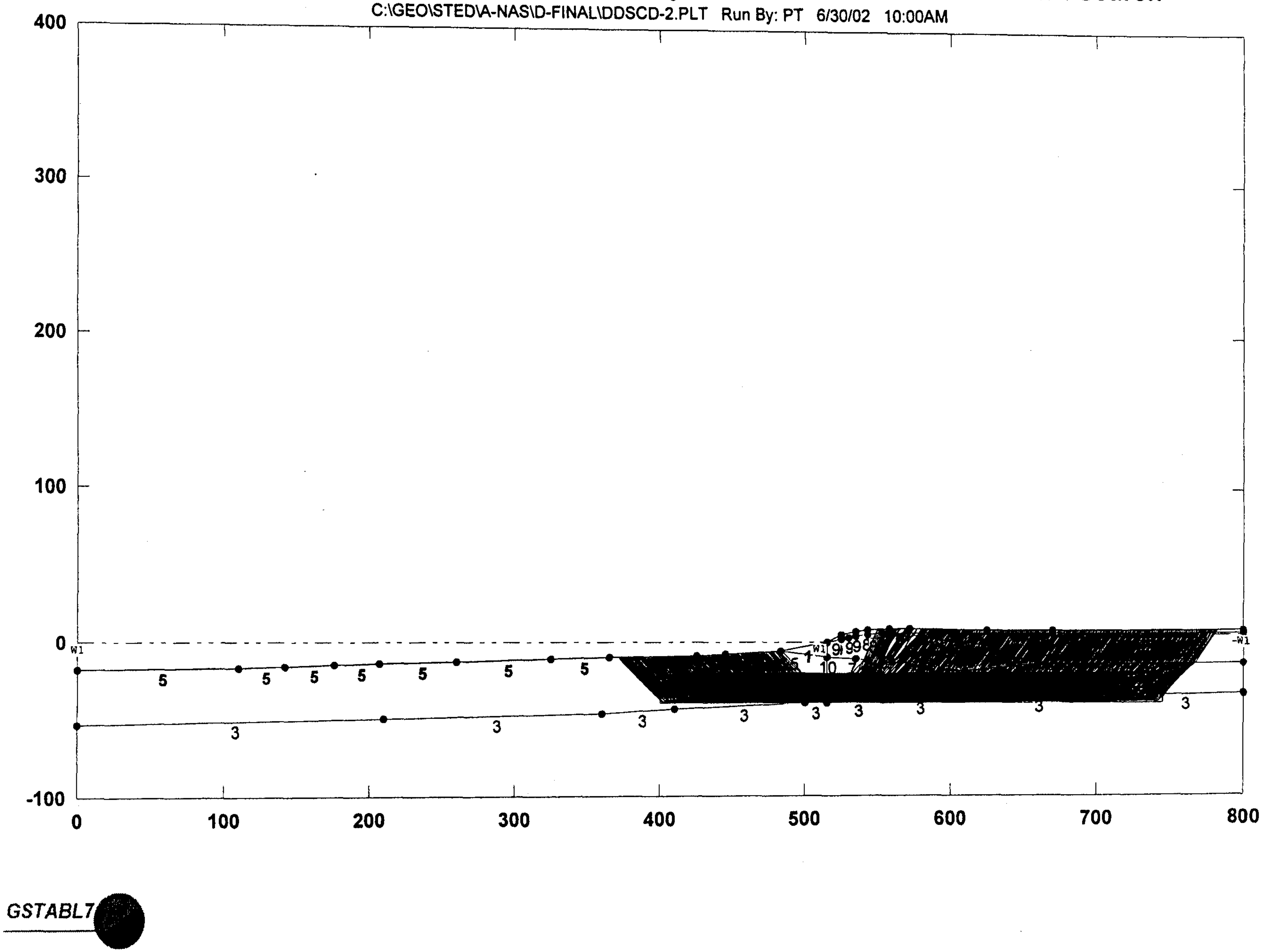
GSTABL7

```
PROFIL  C:\geo\sted\a-nas\d-final\ddscd-1.in Version G7v.2 [GSTABL72.EXE] /O(0,
-100)
e
A-NAS - Section D-D', StoneCol w/Cover  Dynamic Bishop Circular Search
48 19
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 525. 105.1 6
525. 105.1 535. 107.1 6
535. 107.1 543. 108.1 6
543. 108.1 558. 109.1 6
558. 109.1 572. 109.1 6
572. 109.1 625. 108.1 6
625. 108.1 670. 108.1 6
670. 108.1 800. 107.6 6
515.1 100.2 525. 102.1 9
525. 102.1 530. 103.1 9
530. 103.1 535. 104.1 9
535. 104.1 543. 105.1 8
543. 105.1 553. 105.77 8
553. 105.77 558. 106.1 1
515. 90. 515.1 100.2 9
553. 105.77 553.1 87.4 8
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 10
515. 90. 535. 88.67 10
535. 88.67 543. 88.11 7
543. 88.11 553.1 87.4 8
553.1 87.4 560. 87. 2
553.1 87.4 553.2 61.6 8
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 553.2 61.6 3
553.2 61.6 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
```

```
105. 126. 200. 18. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
WATER
1 62.4
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
EQUAKE
0.12 0. 0.
CIRCL2-Bishop circular, search
30 100
300. 450. 500. 700.
0. 6. 0. 0.
```

A-NAS - Section D-D', StoneCol w/Cover Dynamic Janbu Block Failure Surf. Search

C:\GEO\STED\A-NAS\D-FINAL\DDSCD-2.PLT Run By: PT 6/30/02 10:00AM



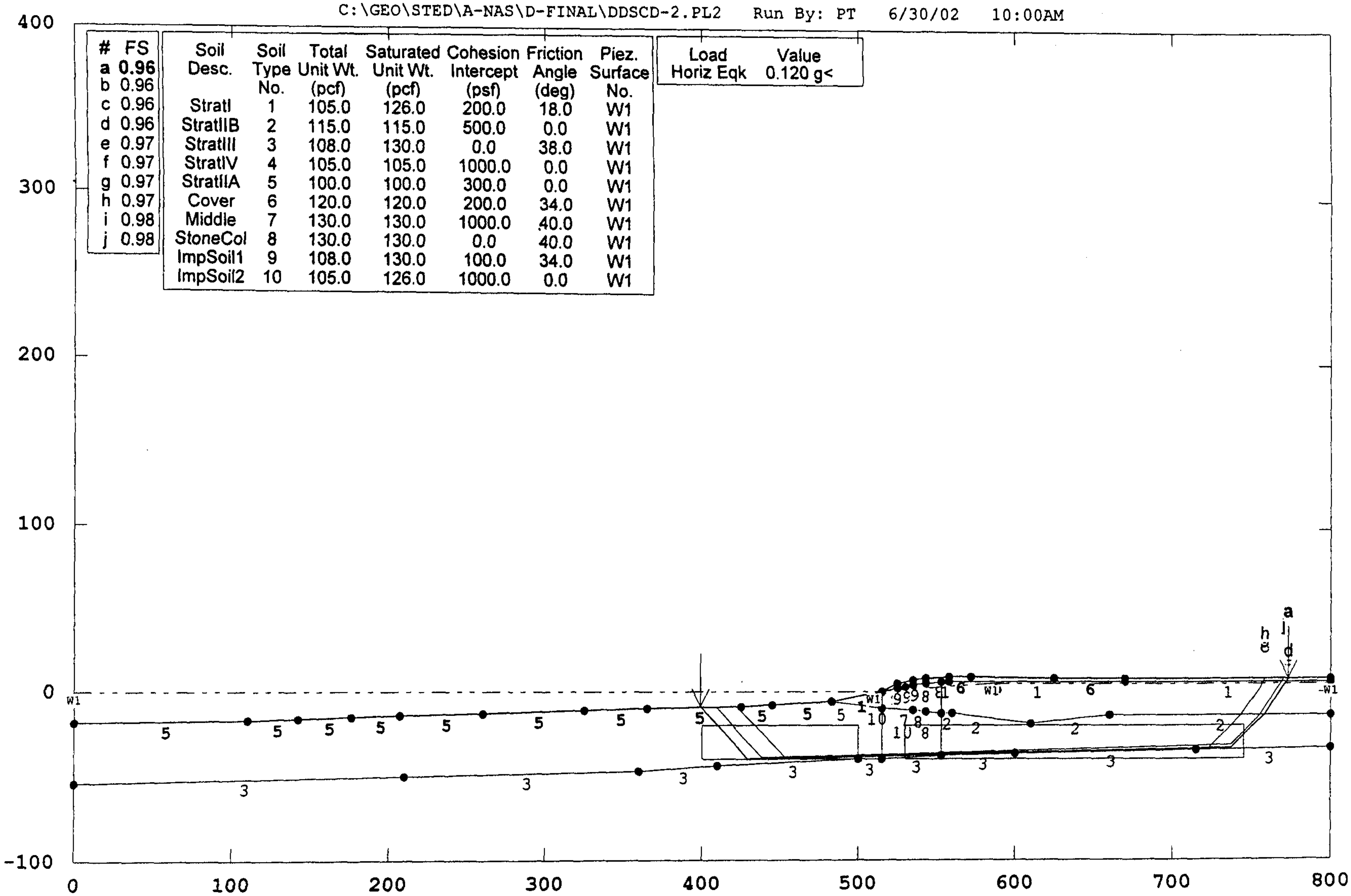
A-NAS - Section D-D', StoneCol w/Cover Dynamic Janbu Block Failure Surf. Search

C:\GEO\STED\A-NAS\D-FINAL\DDSCD-2.PL2 Run By: PT 6/30/02 10:00AM

#	FS
a	**0.96**
b	0.96
c	0.96
d	0.96
e	0.97
f	0.97
g	0.97
h	0.97
i	0.98
j	0.98

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	200.0	18.0	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1000.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Middle	7	130.0	130.0	1000.0	40.0	W1
StoneCol	8	130.0	130.0	0.0	40.0	W1
ImpSoil1	9	108.0	130.0	100.0	34.0	W1
ImpSoil2	10	105.0	126.0	1000.0	0.0	W1

Load	Value
Horiz Eqk	0.120 g<



GSTABL7 v.2 FSmin=0.96

Safety Factors Are Calculated By The Simplified Janbu Method

GSTABL7

```
PROFIL  c:\geo\sted\a-nas\d-final\ddscd-2.in Version G7v.2 [GSTABL72.EXE] /O(0,
-100)
e
A-NAS - Section D-D', StoneCol w/Cover  Dynamic Janbu Block Failure Surf. Search
48 19
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 525. 105.1 6
525. 105.1 535. 107.1 6
535. 107.1 543. 108.1 6
543. 108.1 558. 109.1 6
558. 109.1 572. 109.1 6
572. 109.1 625. 108.1 6
625. 108.1 670. 108.1 6
670. 108.1 800. 107.6 6
515.1 100.2 525. 102.1 9
525. 102.1 530. 103.1 9
530. 103.1 535. 104.1 9
535. 104.1 543. 105.1 8
543. 105.1 553. 105.77 8
553. 105.77 558. 106.1 1
515. 90. 515.1 100.2 9
553. 105.77 553.1 87.4 8
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 10
515. 90. 535. 88.67 10
535. 88.67 543. 88.11 7
543. 88.11 553.1 87.4 8
553.1 87.4 560. 87. 2
553.1 87.4 553.2 61.6 8
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 553.2 61.6 3
553.2 61.6 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
```

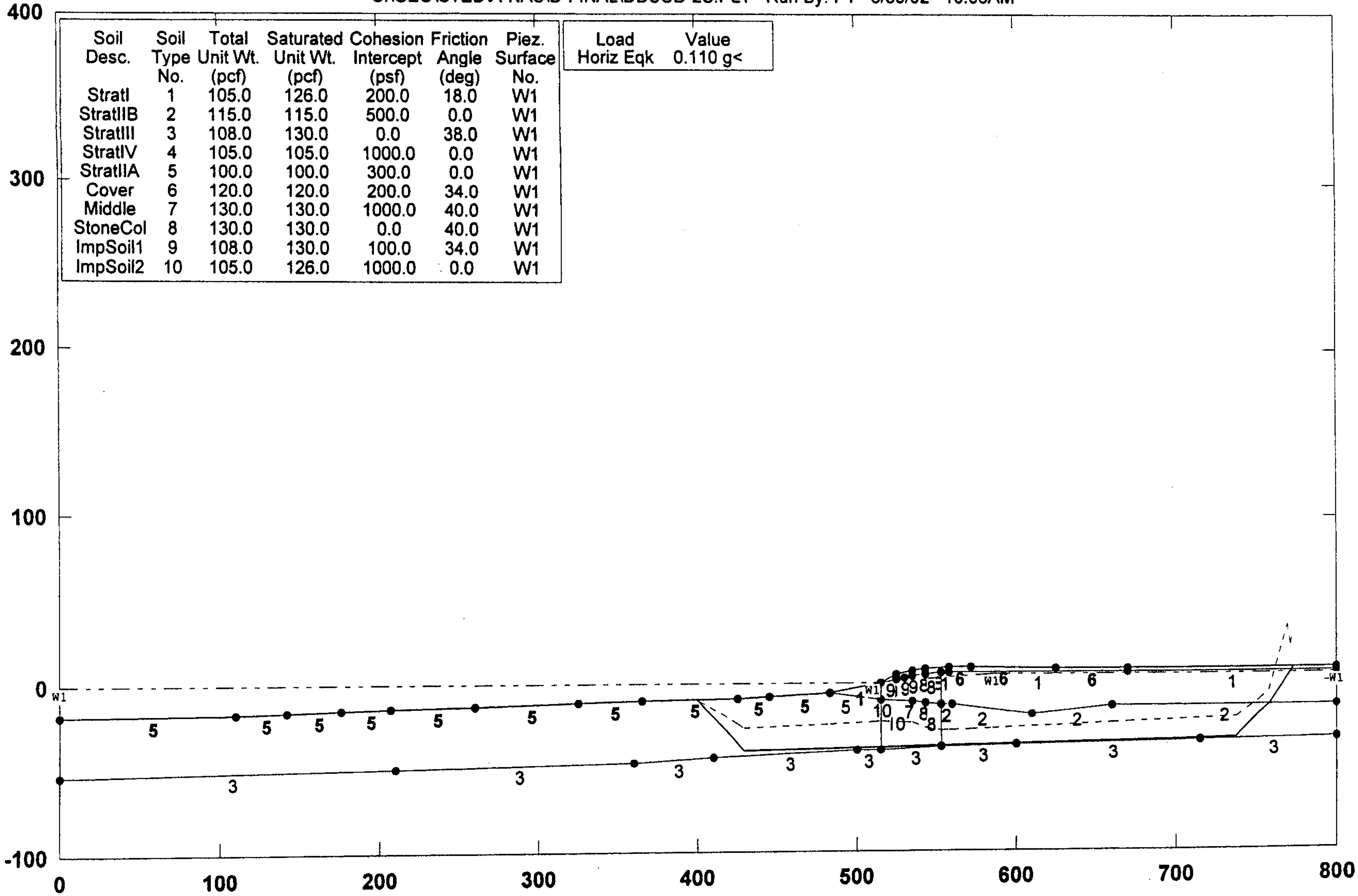
```
105. 126. 200. 18. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
WATER
1 62.4
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
EQUAKE
0.12 0. 0.
BLOCK2
0
4000 2 20.
400. 70. 500. 70. 20.
530. 70. 745. 70. 20.
```

A-NAS - Section D-D', StoneCol w/Cover Dynamic Spencer Block Failure Search

C:\GEO\STED\A-NAS\D-FINAL\DDSCD-2S.PLT Run By: PT 6/30/02 10:06AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	200.0	18.0	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1000.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Middle	7	130.0	130.0	1000.0	40.0	W1
StoneCol	8	130.0	130.0	0.0	40.0	W1
ImpSoil1	9	108.0	130.0	100.0	34.0	W1
ImpSoil2	10	105.0	126.0	1000.0	0.0	W1

Load	Value
Horiz Eqk	0.110 g<



GSTABL7 v.2 FSmin=1.03

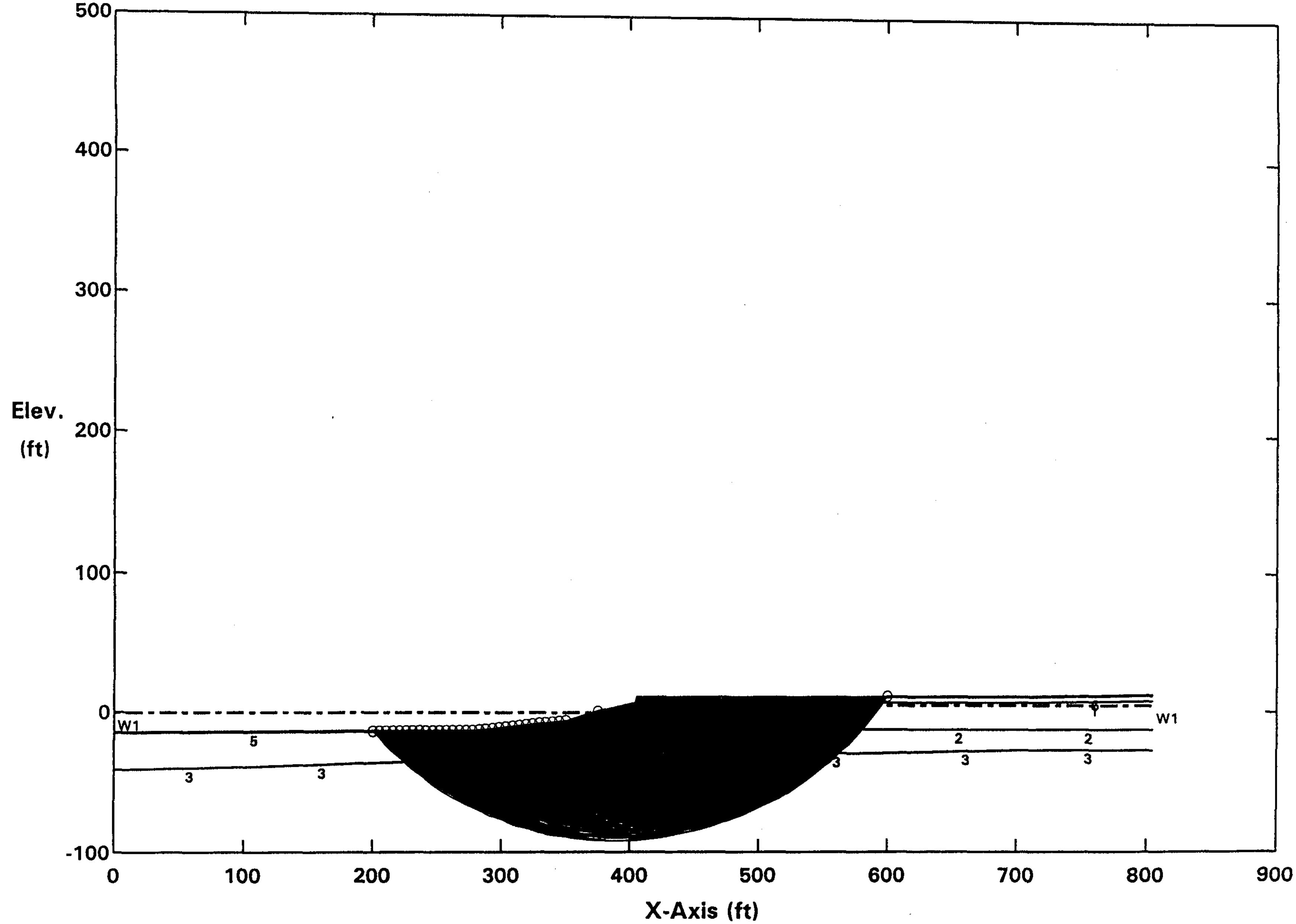
Factor Of Safety Is Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

```
PROFIL  C:\geo\sted\a-nas\d-final\ddscd-2s.in Version G7v.2 [GSTABL72.EXE] /O(0,
-100)
e
A-NAS - Section D-D', StoneCol w/Cover  Dynamic Spencer Block Failure Search
48 19
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 525. 105.1 6
525. 105.1 535. 107.1 6
535. 107.1 543. 108.1 6
543. 108.1 558. 109.1 6
558. 109.1 572. 109.1 6
572. 109.1 625. 108.1 6
625. 108.1 670. 108.1 6
670. 108.1 800. 107.6 6
515.1 100.2 525. 102.1 9
525. 102.1 530. 103.1 9
530. 103.1 535. 104.1 9
535. 104.1 543. 105.1 8
543. 105.1 553. 105.77 8
553. 105.77 558. 106.1 1
515. 90. 515.1 100.2 9
553. 105.77 553.1 87.4 8
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 10
515. 90. 535. 88.67 10
535. 88.67 543. 88.11 7
543. 88.11 553.1 87.4 8
553.1 87.4 560. 87. 2
553.1 87.4 553.2 61.6 8
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 553.2 61.6 3
553.2 61.6 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
```

```
105. 126. 200. 18. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
WATER
1 62.4
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
EQUAKE
0.11 0. 0.
GLEMS
7.
1       Water filled tension crack (0=no,1=yes)
0       Force Distribution (0=Single slice,1=Entire failure surf)
0       Select Method (0=Spencer,1=Morgenstern-Price)
2       ki function (Spencer=1 or 2, M-P=1, 2, 3, 4, or 5=user)
1.000   Lambda Coefficient (adjusts ki, 0.4 to 1.0)
0       Trial Lambda Adjustment option (0=no, 1=yes)
SURFAC
0
10
398.65 90.56
400.54 88.66
414.68 74.52
428.83 60.38
737.17 65.64
751.31 79.79
757.52 86.
769.28 102.18
771.84 105.71
772.9 107.7
EXECUT
```

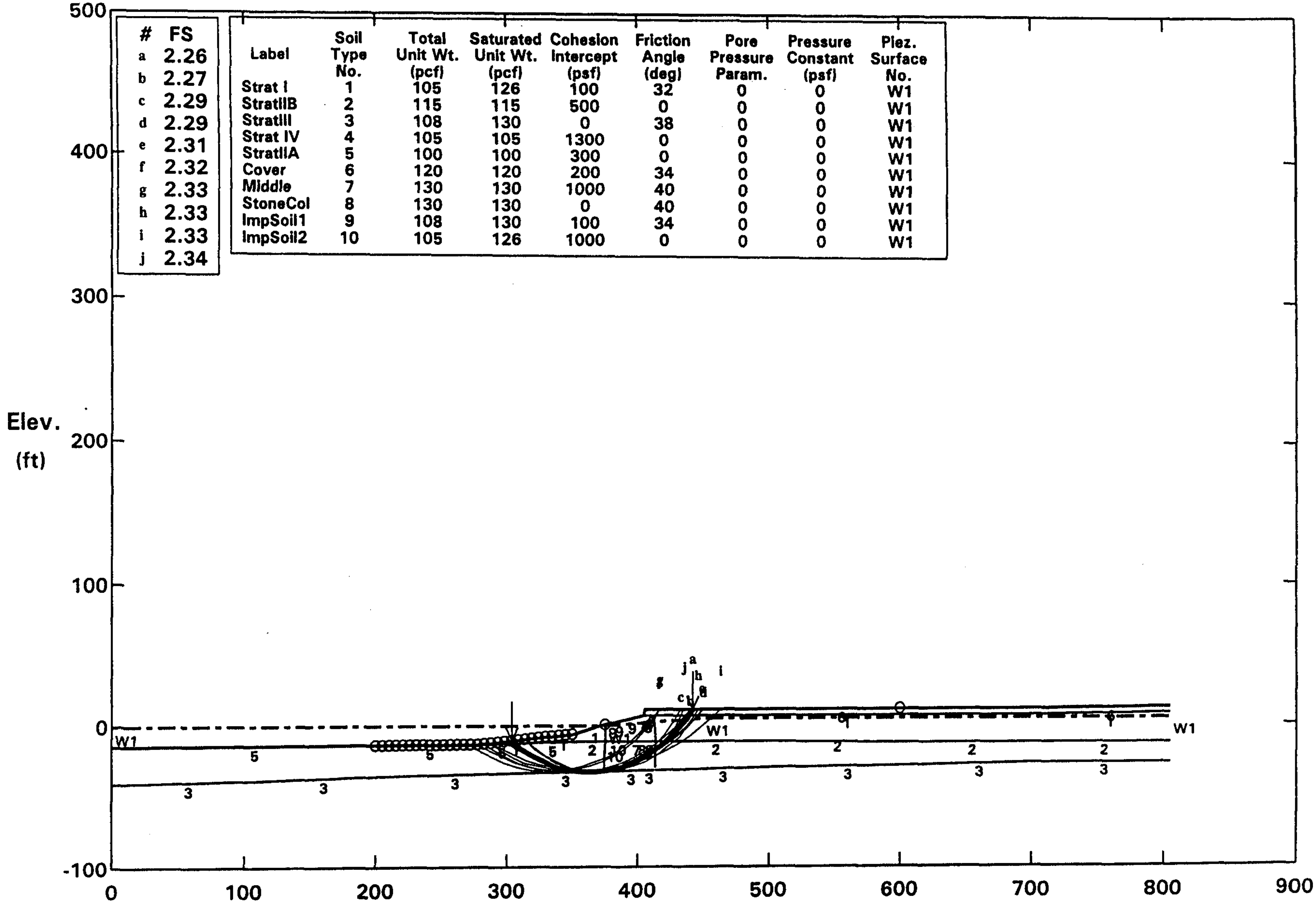
A-NAS - Section F-F', StoneCol w/Cover Static Long Term-Bishop Circular Search
All surfaces evaluated. C:FFSCS-LT.PLT By: BH 06-30-02 11:14am



Factors Of Safety Calculated By The Modified Bishop Method

A-NAS - Section F-F', StoneCol w/Cover Static Long Term-Bishop Circular Search

Ten Most Critical. C:FFSCS-LT.PLT By: BH 06-30-02 11:14am



#	FS
a	2.26
b	2.27
c	2.29
d	2.29
e	2.31
f	2.32
g	2.33
h	2.33
i	2.33
j	2.34

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Strat I	1	105	126	100	32	0	0	W1
StratIIB	2	115	115	500	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
Strat IV	4	105	105	1300	0	0	0	W1
StratIIA	5	100	100	300	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
Middle	7	130	130	1000	40	0	0	W1
StoneCol	8	130	130	0	40	0	0	W1
ImpSoil1	9	108	130	100	34	0	0	W1
ImpSoil2	10	105	126	1000	0	0	0	W1

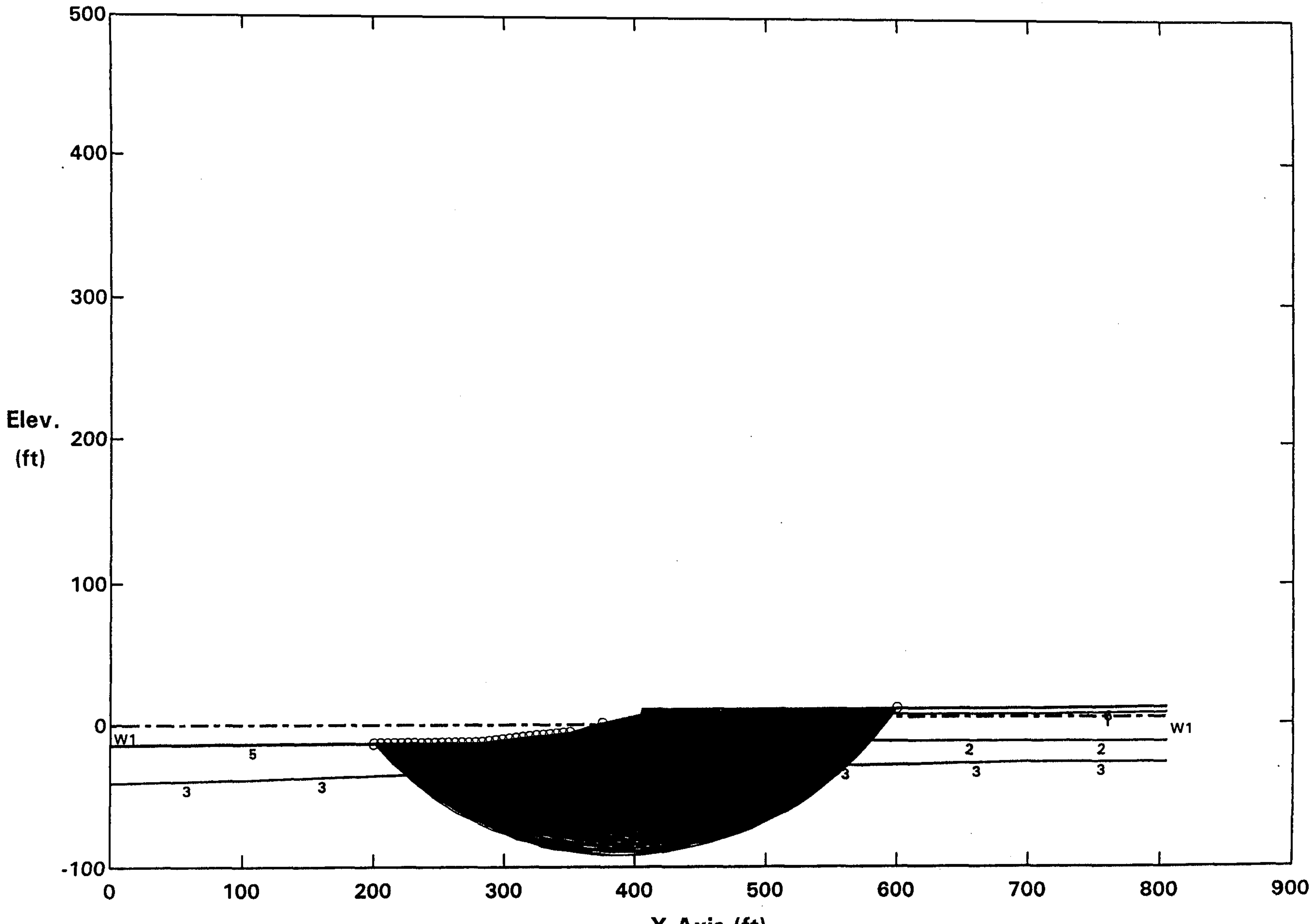
PCSTABL5M FSmin = 2.26 X-Axis (ft)

Factors Of Safety Cal[illegible]d By The Modified Bishop Method

```
PROFIL       C:\GEO\STED\A-NAS\F-FINAL\FFSCS-LT.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section F-F', StoneCol w/Cover  Static Long Term-Bishop Circular Search
39 11
0. 85.5 205. 86.5 5
205. 86.5 278. 87. 5
278. 87. 327. 92. 1
327. 92. 350. 94. 1
350. 94. 375. 100.5 1
375. 100.5 385. 103. 9
385. 103. 395. 105.05 9
395. 105.05 405. 107.1 8
405. 107.1 405.1 111.1 6
405.1 111.1 705. 111.1 6
705. 111.1 805. 112.1 6
374.9 88.7 375. 100.5 9
405. 107.1 413. 107.1 8
413. 107.1 705. 107.1 1
413. 107.1 413.1 90. 8
705. 107.1 805. 108.1 1
278. 87. 305. 88. 5
305. 88. 355. 88.5 5
355. 88.5 374.9 88.7 2
374.8 67.4 374.9 88.7 10
374.9 88.7 395. 88.9 10
395. 88.9 403. 88.98 7
403. 88.98 405. 89. 8
405. 89. 413.1 88.96 8
413.1 88.96 505. 88.5 2
413.1 88.96 413.2 70. 8
505. 88.5 605. 88. 2
605. 88. 705. 88. 2
705. 88. 805. 88. 2
0. 59. 105. 61.5 3
105. 61.5 205. 64.5 3
205. 64.5 305. 66. 3
305. 66. 374.8 67.4 3
374.8 67.4 405. 68. 3
405. 68. 413. 68.2 3
413.2 68.2 505. 70. 3
505. 70. 605. 71. 3
605. 71. 705. 73. 3
705. 73. 805. 73.5 3
SOIL  Strat I StratIIBStratIIIStrat IVStratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1300. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
WATER
```

```
1 62.4
4
0. 100.
375. 100.
450. 105.
805. 105.
CIRCL2-Bishop circular, search
30 100
200. 350. 375. 600. 0. 10. 0. 0.
```

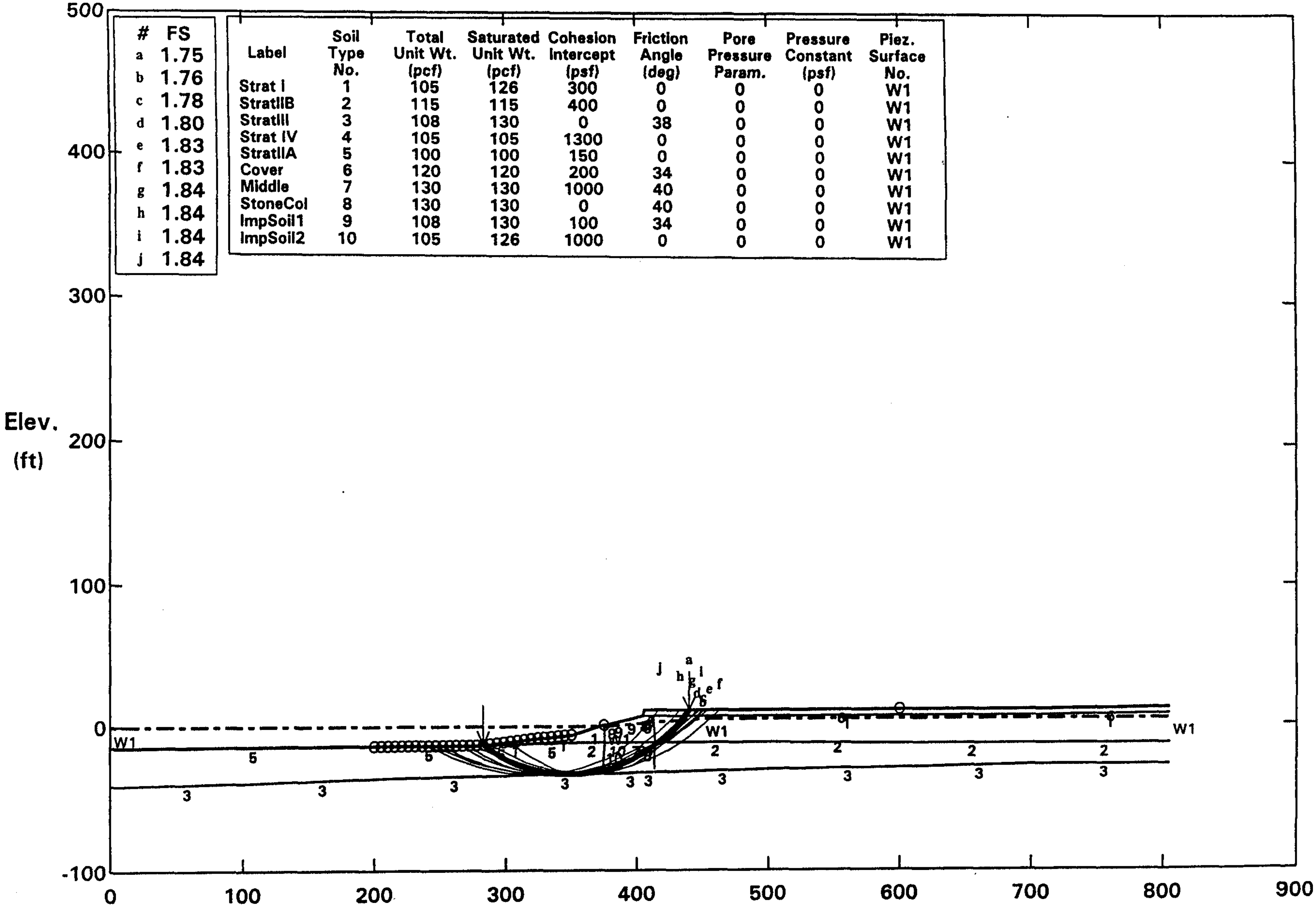
A-NAS - Section F-F', StoneCol w/Cover Static Post-EQ, Bishop Circular Search
All surfaces evaluated. C:FFSCS-PE.PLT By: BH 06-30-02 11:08am



Factors Of Safety Calculated By The Modified Bishop Method

A-NAS - Section F-F', StoneCol w/Cover Static Post-EQ, Bishop Circular Search

Ten Most Critical. C:FFSCS-PE.PLT By: BH 06-30-02 11:08am



#	FS
a	1.75
b	1.76
c	1.78
d	1.80
e	1.83
f	1.83
g	1.84
h	1.84
i	1.84
j	1.84

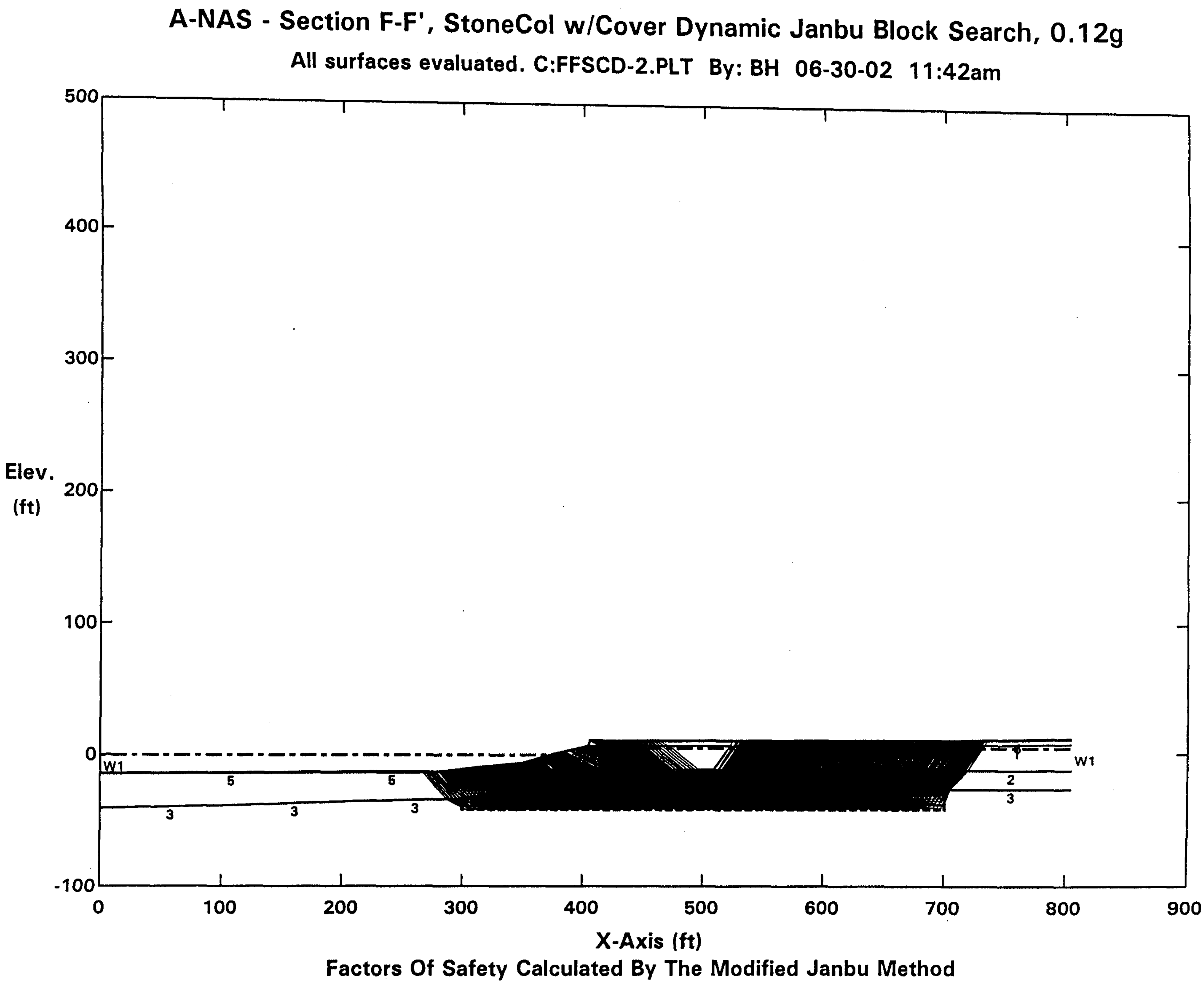
Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Strat I	1	105	126	300	0	0	0	W1
StratIIB	2	115	115	400	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
Strat IV	4	105	105	1300	0	0	0	W1
StratIIA	5	100	100	150	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
Middle	7	130	130	1000	40	0	0	W1
StoneCol	8	130	130	0	40	0	0	W1
ImpSoil1	9	108	130	100	34	0	0	W1
ImpSoil2	10	105	126	1000	0	0	0	W1

PCSTABL5M FSmin = 1.75 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

```
PROFIL        C:\GEO\STED\A-NAS\F-FINAL\FFSCS-PE.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section F-F', StoneCol w/Cover  Static Post-EQ, Bishop Circular Search
39 11
0. 85.5 205. 86.5 5
205. 86.5 278. 87. 5
278. 87. 327. 92. 1
327. 92. 350. 94. 1
350. 94. 375. 100.5 1
375. 100.5 385. 103. 9
385. 103. 395. 105.05 9
395. 105.05 405. 107.1 8
405. 107.1 405.1 111.1 6
405.1 111.1 705. 111.1 6
705. 111.1 805. 112.1 6
374.9 88.7 375. 100.5 9
405. 107.1 413. 107.1 8
413. 107.1 705. 107.1 1
413. 107.1 413.1 90. 8
705. 107.1 805. 108.1 1
278. 87. 305. 88. 5
305. 88. 355. 88.5 5
355. 88.5 374.9 88.7 2
374.8 67.4 374.9 88.7 10
374.9 88.7 395. 88.9 10
395. 88.9 403. 88.98 7
403. 88.98 405. 89. 8
405. 89. 413.1 88.96 8
413.1 88.96 505. 88.5 2
413.1 88.96 413.2 70. 8
505. 88.5 605. 88. 2
605. 88. 705. 88. 2
705. 88. 805. 88. 2
0. 59. 105. 61.5 3
105. 61.5 205. 64.5 3
205. 64.5 305. 66. 3
305. 66. 374.8 67.4 3
374.8 67.4 405. 68. 3
405. 68. 413. 68.2 3
413.2 68.2 505. 70. 3
505. 70. 605. 71. 3
605. 71. 705. 73. 3
705. 73. 805. 73.5 3
SOIL  Strat I StratIIBStratIIIStrat IVStratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
105. 126. 300. 0. 0. 0. 1
115. 115. 400. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1300. 0. 0. 0. 1
100. 100. 150. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
WATER
```

```
1 62.4
4
0. 100.
375. 100.
450. 105.
805. 105.
CIRCL2-Bishop circular, search
30 100
200. 350. 375. 600. 0. 10. 0. 0.
```



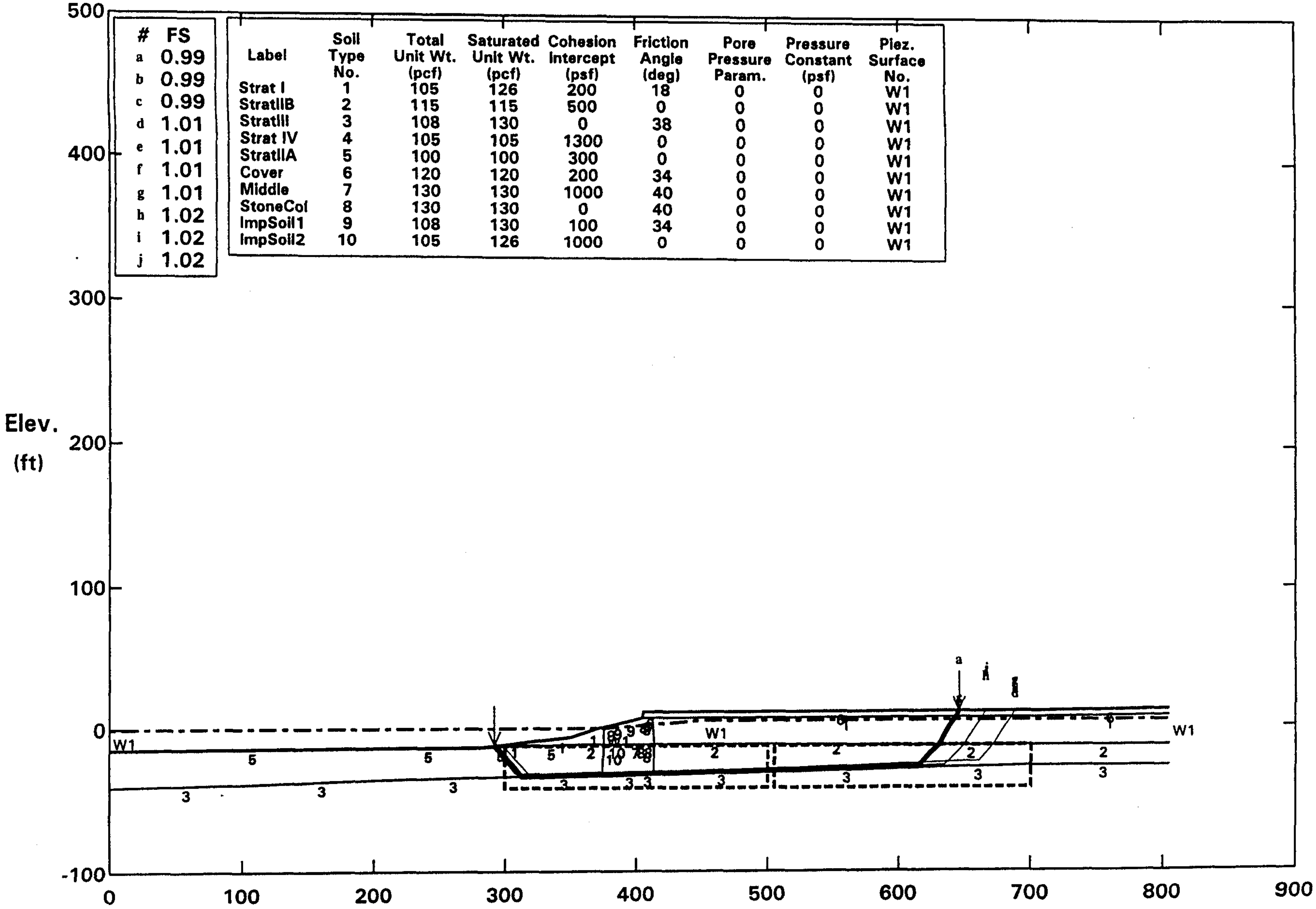
A-NAS - Section F-F', StoneCol w/Cover Dynamic Janbu Block Search, 0.12g
All surfaces evaluated. C:FFSCD-2.PLT By: BH 06-30-02 11:42am
Elev. (ft)
500
400
300
200
100
0
-100
W1
5
5
3
3
3
2
3
W1
0
100
200
300
400
500
600
700
800
900
X-Axis (ft)
Factors Of Safety Calculated By The Modified Janbu Method

A-NAS - Section F-F', StoneCol w/Cover Dynamic Janbu Block Search, 0.12g

Ten Most Critical. C:FFSCD-2.PLT By: BH 06-30-02 11:42am

#	FS
a	0.99
b	0.99
c	0.99
d	1.01
e	1.01
f	1.01
g	1.01
h	1.02
i	1.02
j	1.02

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Strat I	1	105	126	200	18	0	0	W1
StratIIB	2	115	115	500	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
Strat IV	4	105	105	1300	0	0	0	W1
StratIIA	5	100	100	300	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
Middle	7	130	130	1000	40	0	0	W1
StoneCol	8	130	130	0	40	0	0	W1
ImpSoil1	9	108	130	100	34	0	0	W1
ImpSoil2	10	105	126	1000	0	0	0	W1



PCSTABL5M FSmin = 0.99 X-Axis (ft)

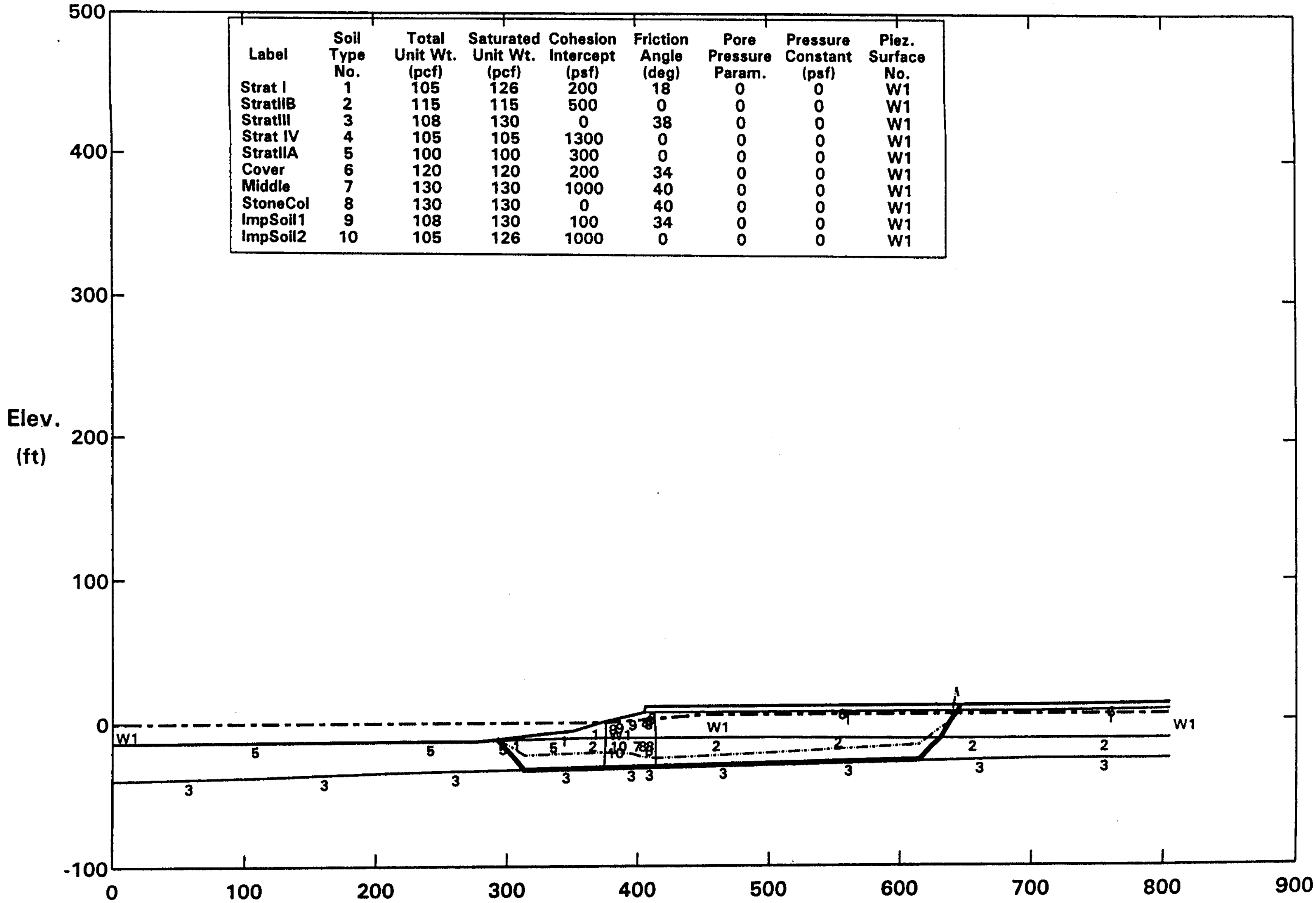
Factors Of Safety Calculated By The Modified Janbu Method

```
PROFIL       C:\GEO\STED\A-NAS\F-FINAL\FFSCD-2.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section F-F', StoneCol w/Cover  Dynamic Janbu Block Search, 0.12g
39 11
0. 85.5 205. 86.5 5
205. 86.5 278. 87. 5
278. 87. 327. 92. 1
327. 92. 350. 94. 1
350. 94. 375. 100.5 1
375. 100.5 385. 103. 9
385. 103. 395. 105.05 9
395. 105.05 405. 107.1 8
405. 107.1 405.1 111.1 6
405.1 111.1 705. 111.1 6
705. 111.1 805. 112.1 6
374.9 88.7 375. 100.5 9
405. 107.1 413. 107.1 8
413. 107.1 705. 107.1 1
413. 107.1 413.1 90. 8
705. 107.1 805. 108.1 1
278. 87. 305. 88. 5
305. 88. 355. 88.5 5
355. 88.5 374.9 88.7 2
374.8 67.4 374.9 88.7 10
374.9 88.7 395. 88.9 10
395. 88.9 403. 88.98 7
403. 88.98 405. 89. 8
405. 89. 413.1 88.96 8
413.1 88.96 505. 88.5 2
413.1 88.96 413.2 70. 8
505. 88.5 605. 88. 2
605. 88. 705. 88. 2
705. 88. 805. 88. 2
0. 59. 105. 61.5 3
105. 61.5 205. 64.5 3
205. 64.5 305. 66. 3
305. 66. 374.8 67.4 3
374.8 67.4 405. 68. 3
405. 68. 413. 68.2 3
413.2 68.2 505. 70. 3
505. 70. 605. 71. 3
605. 71. 705. 73. 3
705. 73. 805. 73.5 3
SOIL  Strat I StratIIBStratIIIStrat IVStratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
105. 126. 200. 18. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1300. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
WATER
```

```
1 62.4
4
0. 100.
375. 100.
450. 105.
805. 105.
EQUAKE
0.12 0. 0.
BLOCK2-Rankine block, search
0
4000 2 15.
300. 73. 500. 73. 30.
505. 73. 700. 73. 30.
```

A-NAS - Section F-F', StoneCol w/Cover Dynamic Spencer Block Search, 0.12g

Surface #1-FFSCD-2.OUT. C:FFSCD-2S.PLT By: BH 06-30-02 11:44am



Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Strat I	1	105	126	200	18	0	0	W1
StratIIB	2	115	115	500	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
Strat IV	4	105	105	1300	0	0	0	W1
StratIIA	5	100	100	300	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
Middle	7	130	130	1000	40	0	0	W1
StoneCol	8	130	130	0	40	0	0	W1
ImpSoil1	9	108	130	100	34	0	0	W1
ImpSoil2	10	105	126	1000	0	0	0	W1

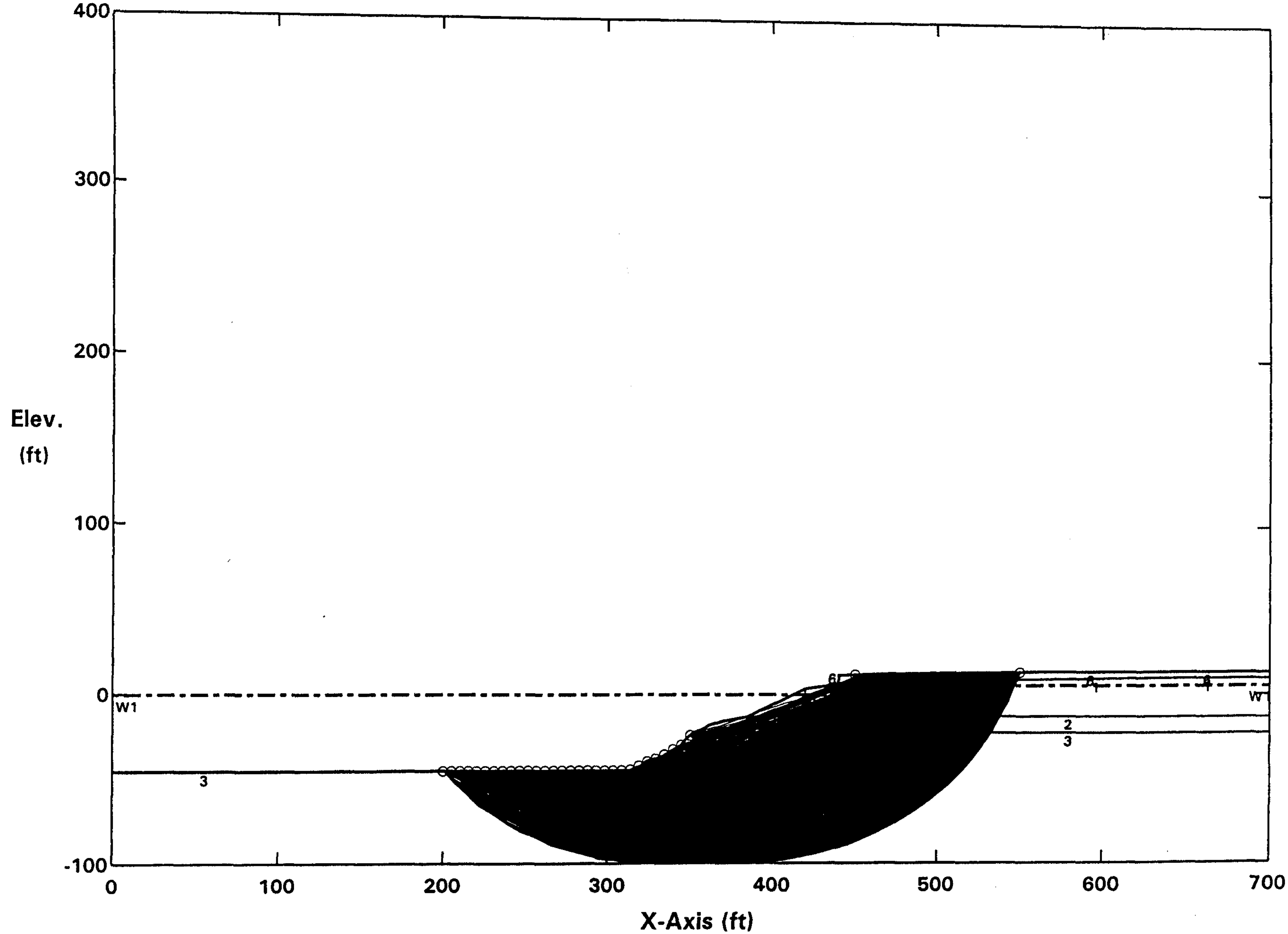
PCSTABL5M FS = 1.01 Theta = 1.56 X-Axis (ft)

Factors Of Safety Calculated By Spencer`s Method of Slices

```
PROFIL        C:\GEO\STED\A-NAS\F-FINAL\FFSCD-2S.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section F-F', StoneCol w/Cover  Dynamic Spencer Block Search, 0.12g
39 11
0. 85.5 205. 86.5 5
205. 86.5 278. 87. 5
278. 87. 327. 92. 1
327. 92. 350. 94. 1
350. 94. 375. 100.5 1
375. 100.5 385. 103. 9
385. 103. 395. 105.05 9
395. 105.05 405. 107.1 8
405. 107.1 405.1 111.1 6
405.1 111.1 705. 111.1 6
705. 111.1 805. 112.1 6
374.9 88.7 375. 100.5 9
405. 107.1 413. 107.1 8
413. 107.1 705. 107.1 1
413. 107.1 413.1 90. 8
705. 107.1 805. 108.1 1
278. 87. 305. 88. 5
305. 88. 355. 88.5 5
355. 88.5 374.9 88.7 2
374.8 67.4 374.9 88.7 10
374.9 88.7 395. 88.9 10
395. 88.9 403. 88.98 7
403. 88.98 405. 89. 8
405. 89. 413.1 88.96 8
413.1 88.96 505. 88.5 2
413.1 88.96 413.2 70. 8
505. 88.5 605. 88. 2
605. 88. 705. 88. 2
705. 88. 805. 88. 2
0. 59. 105. 61.5 3
105. 61.5 205. 64.5 3
205. 64.5 305. 66. 3
305. 66. 374.8 67.4 3
374.8 67.4 405. 68. 3
405. 68. 413. 68.2 3
413.2 68.2 505. 70. 3
505. 70. 605. 71. 3
605. 71. 705. 73. 3
705. 73. 805. 73.5 3
SOIL  Strat I StratIIBStratIIIStrat IVStratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
105. 126. 200. 18. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1300. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
WATER
```

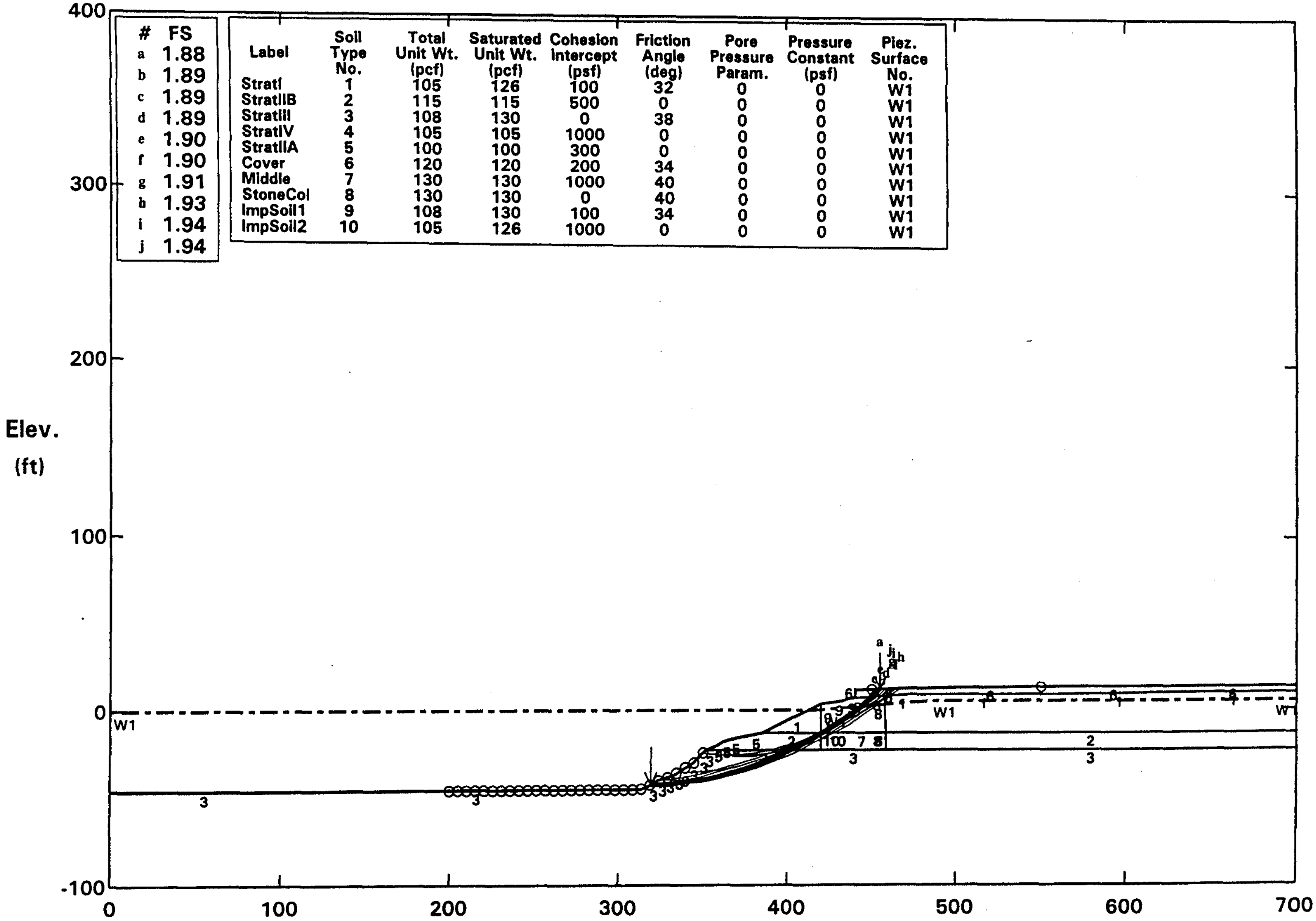
```
1 62.4
4
0. 100.
375. 100.
450. 105.
805. 105.
EQUAKE
0.12 0. 0.
SPENCR
10.
SURFAC #1-FFSCD-2.OUT
10
291.44 88.37
292.58 87.54
302.71 77.41
313.32 66.8
614.63 72.12
625.23 82.73
630.51 88.
639.33 100.14
644.39 107.1
646.51 111.1
EXECUT
```

A-NAS - Section G-G', StoneCol w/Cover Static Long-Term Bishop Circular Search
All surfaces evaluated. C:GGSCS-LT.PLT By: P.T. 06-30-02 12:05pm



Factors Of Safety Calculated By The Modified Bishop Method

A-NAS - Section G-G', StoneCol w/Cover Static Long-Term Bishop Circular Search
Ten Most Critical. C:GGSCS-LT.PLT By: P.T. 06-30-02 12:05pm



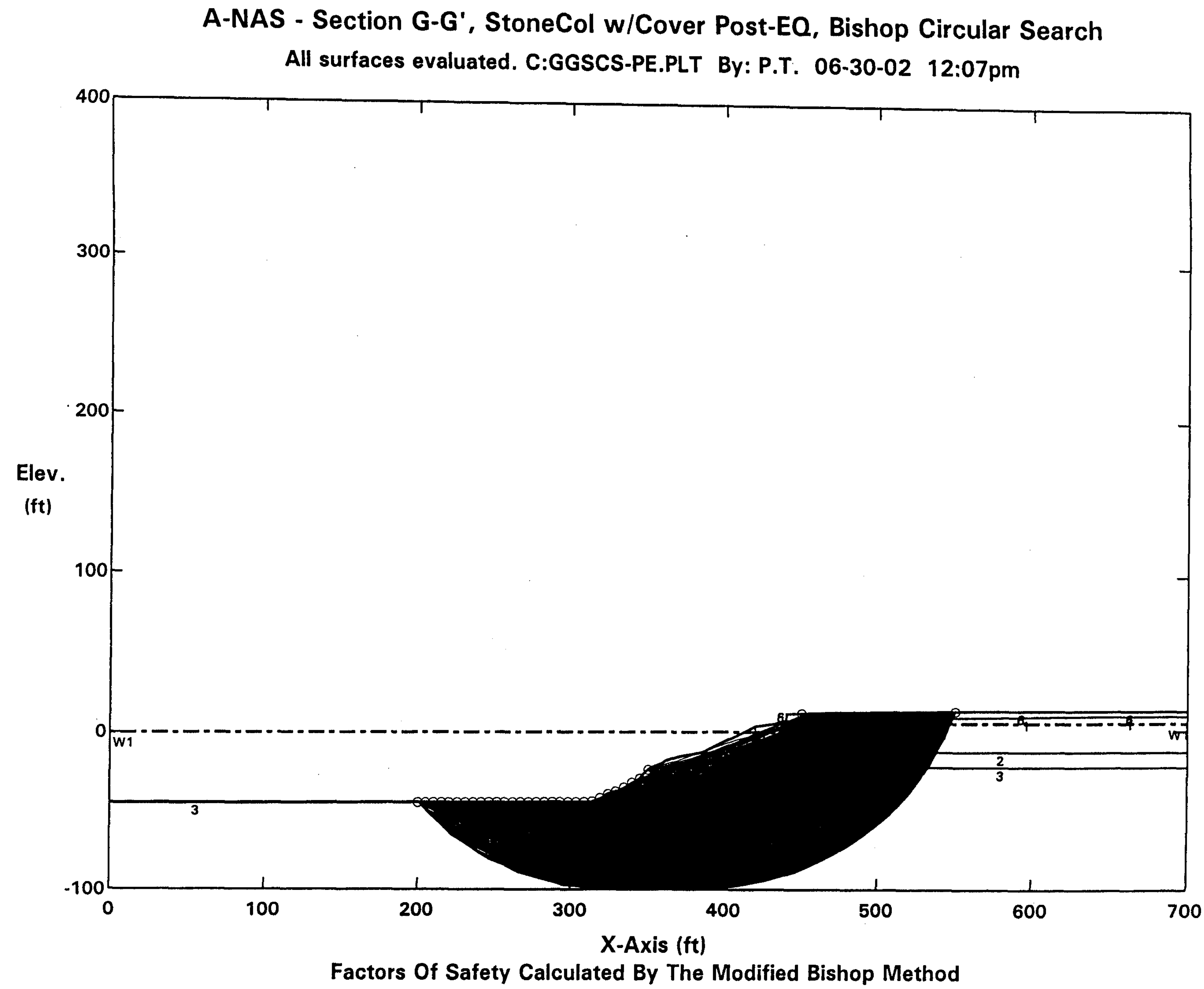
#	FS
a	1.88
b	1.89
c	1.89
d	1.89
e	1.90
f	1.90
g	1.91
h	1.93
i	1.94
j	1.94

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	100	32	0	0	W1
StratIIB	2	115	115	500	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1000	0	0	0	W1
StratIIA	5	100	100	300	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
Middle	7	130	130	1000	40	0	0	W1
StoneCol	8	130	130	0	40	0	0	W1
ImpSoil1	9	108	130	100	34	0	0	W1
ImpSoil2	10	105	126	1000	0	0	0	W1

PCSTABL5M FSmin = 1.88 X-Axis (ft)
Factors Of Safety Calc[illegible]d By The Modified Bishop Method

```
PROFIL       C:\GEO\STED\A-NAS\G-FINAL\GGSCS-LT.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section G-G', StoneCol w/Cover  Static Long-Term Bishop Circular Search
40 23
0. 54. 110. 54. 3
110. 54. 313. 55. 3
313. 55. 320. 58. 3
320. 58. 324. 60. 3
324. 60. 330. 62. 3
330. 62. 334. 64. 3
334. 64. 337. 66. 3
337. 66. 345. 70. 3
345. 70. 348. 74. 3
348. 74. 352. 77. 3
352. 77. 359. 80. 5
359. 80. 361. 81. 5
361. 81. 370. 84. 5
370. 84. 385. 87. 5
385. 87. 420. 103.1 1
420. 103.1 433. 105.1 9
433. 105.1 435. 106.1 9
435. 106.1 440. 107.1 9
440. 107.1 440.1 111.1 6
440.1 111.1 472. 112.1 6
472. 112.1 560. 112.7 6
560. 112.7 625. 113.1 6
625. 113.1 700. 113.1 6
419.9 87. 420. 103.1 9
440. 107.1 458. 107.7 8
458. 107.7 472. 108.1 1
458. 107.7 458.1 87. 8
472. 108.1 560. 108.7 1
560. 108.7 625. 109.1 1
625. 109.1 700. 109.1 1
385. 87. 420. 87. 2
419.8 77. 419.9 87. 10
419.9 87. 440. 87. 10
440. 87. 448. 87. 7
448. 87. 458.1 87. 8
458.1 87. 700. 87. 2
458.1 87. 458.2 77. 8
352. 77. 419.8 77. 3
419.8 77. 458.2 77. 3
458.2 77. 700. 77. 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
```

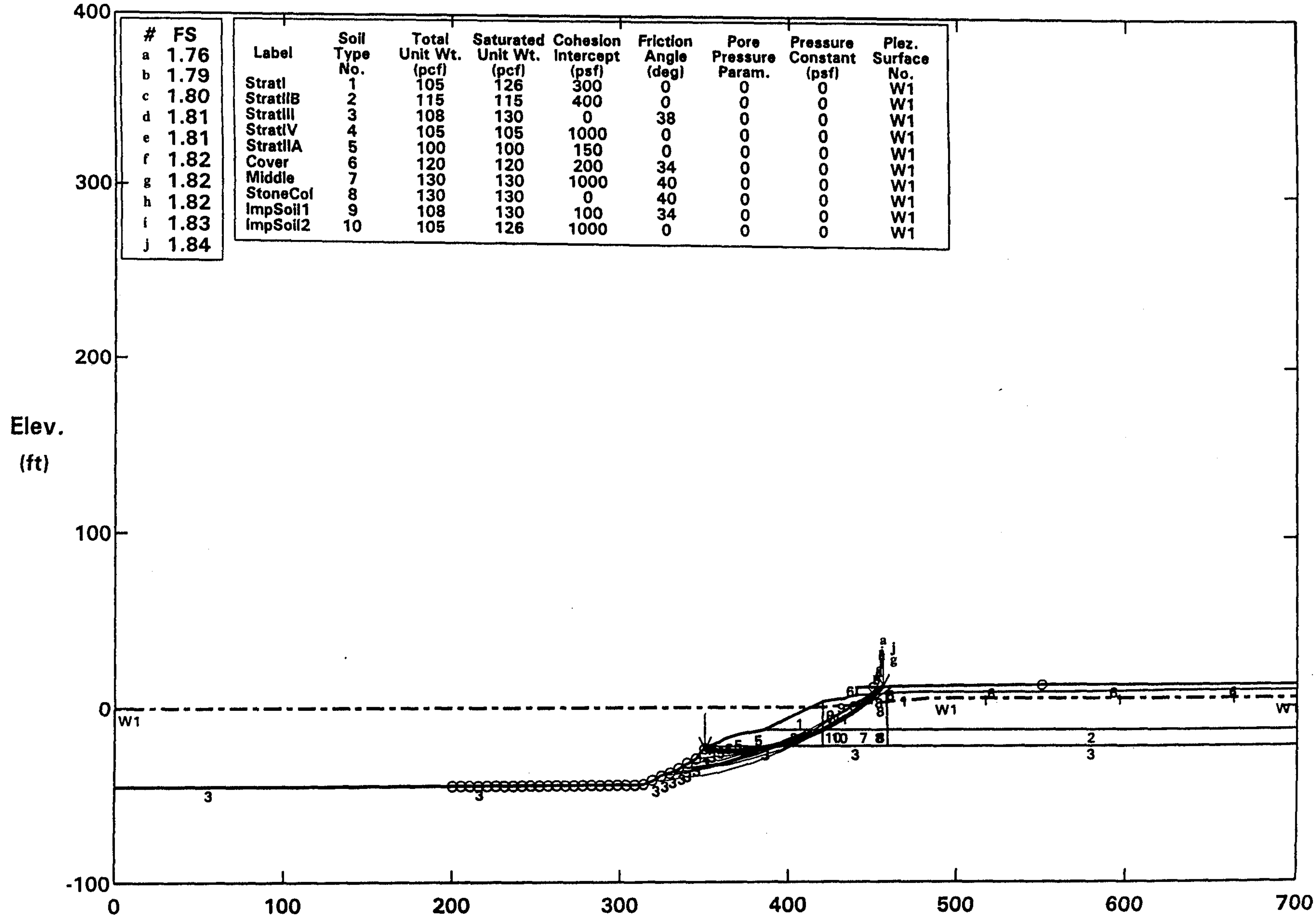
```
WATER
1 62.4
4
0. 100.
420. 100.
485. 105.
700. 105.
CIRCL2-Bishop circular, search
30 100
200. 350. 450. 550. 0. 10. 0. 0.
```



A-NAS - Section G-G', StoneCol w/Cover Post-EQ, Bishop Circular Search
All surfaces evaluated. C:GGSCS-PE.PLT By: P.T. 06-30-02 12:07pm
Elev. (ft)
400
300
200
100
0
-100
0
100
200
300
400
500
600
700
W1
3
2
3
X-Axis (ft)
Factors Of Safety Calculated By The Modified Bishop Method

A-NAS - Section G-G', StoneCol w/Cover Post-EQ, Bishop Circular Search

Ten Most Critical. C:GGSCS-PE.PLT By: P.T. 06-30-02 12:07pm



#	FS
a	1.76
b	1.79
c	1.80
d	1.81
e	1.81
f	1.82
g	1.82
h	1.82
i	1.83
j	1.84

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	300	0	0	0	W1
StratIIB	2	115	115	400	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1000	0	0	0	W1
StratIIA	5	100	100	150	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
Middle	7	130	130	1000	40	0	0	W1
StoneCol	8	130	130	0	40	0	0	W1
ImpSoil1	9	108	130	100	34	0	0	W1
ImpSoil2	10	105	126	1000	0	0	0	W1

PCSTABL5M FSmin = 1.76 X-Axis (ft)

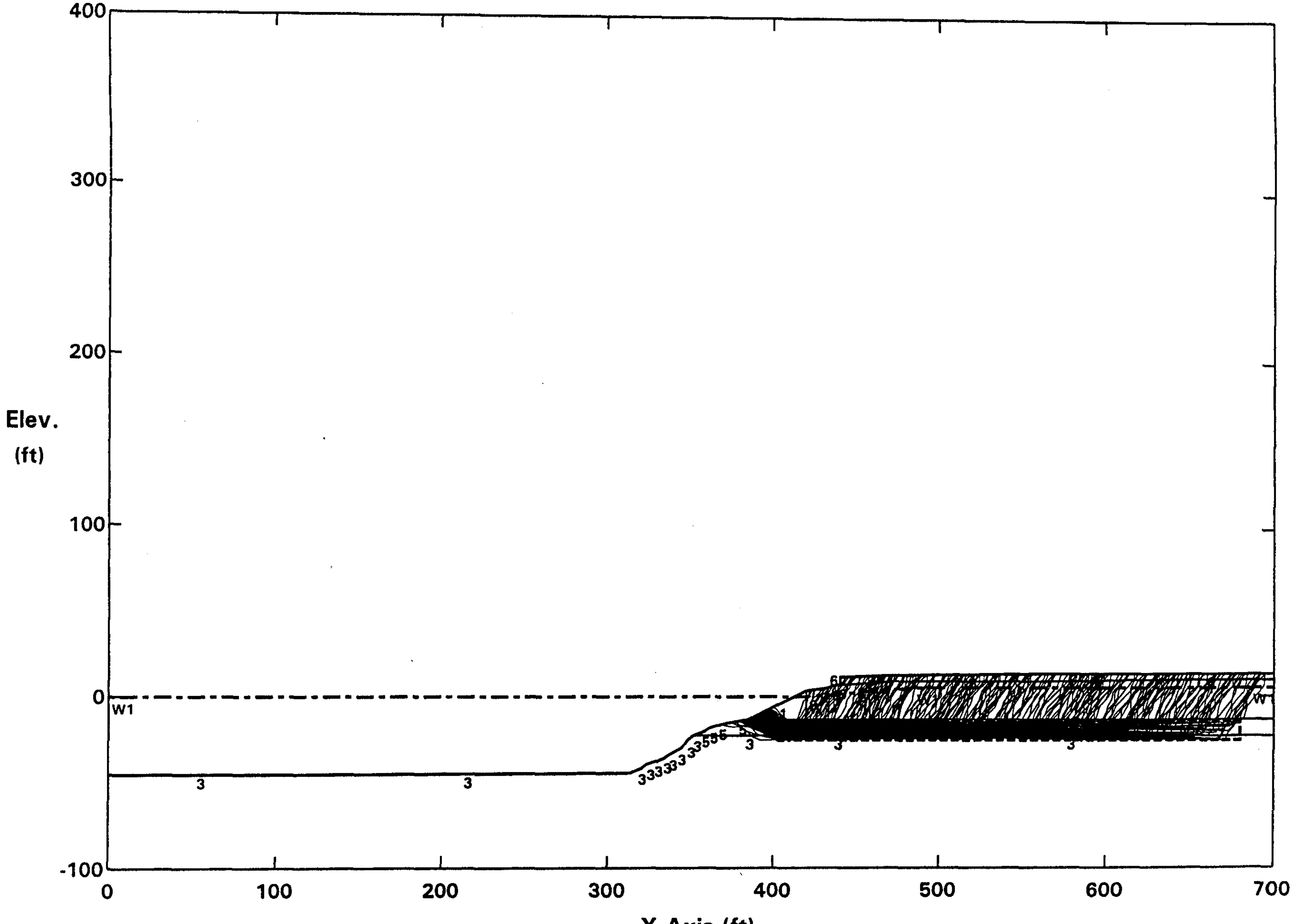
Factors Of Safety Ca[illegible]d By The Modified Bishop Method

```
PROFIL        C:\GEO\STED\A-NAS\G-FINAL\GGSCS-PE.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section G-G', StoneCol w/Cover  Post-EQ, Bishop Circular Search
40 23
0. 54. 110. 54. 3
110. 54. 313. 55. 3
313. 55. 320. 58. 3
320. 58. 324. 60. 3
324. 60. 330. 62. 3
330. 62. 334. 64. 3
334. 64. 337. 66. 3
337. 66. 345. 70. 3
345. 70. 348. 74. 3
348. 74. 352. 77. 3
352. 77. 359. 80. 5
359. 80. 361. 81. 5
361. 81. 370. 84. 5
370. 84. 385. 87. 5
385. 87. 420. 103.1 1
420. 103.1 433. 105.1 9
433. 105.1 435. 106.1 9
435. 106.1 440. 107.1 9
440. 107.1 440.1 111.1 6
440.1 111.1 472. 112.1 6
472. 112.1 560. 112.7 6
560. 112.7 625. 113.1 6
625. 113.1 700. 113.1 6
419.9 87. 420. 103.1 9
440. 107.1 458. 107.7 8
458. 107.7 472. 108.1 1
458. 107.7 458.1 87. 8
472. 108.1 560. 108.7 1
560. 108.7 625. 109.1 1
625. 109.1 700. 109.1 1
385. 87. 420. 87. 2
419.8 77. 419.9 87. 10
419.9 87. 440. 87. 10
440. 87. 448. 87. 7
448. 87. 458.1 87. 8
458.1 87. 700. 87. 2
458.1 87. 458.2 77. 8
352. 77. 419.8 77. 3
419.8 77. 458.2 77. 3
458.2 77. 700. 77. 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
105. 126. 300. 0. 0. 0. 1
115. 115. 400. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 150. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
```

```
WATER
1 62.4
4
0. 100.
420. 100.
485. 105.
700. 105.
CIRCL2-Bishop circular, search
30 100
200. 350. 450. 550. 0. 10. 0. 0.
```

A-NAS - Section G-G', StoneCol w/Cover Dynamic Janbu Block Search, 0.15g

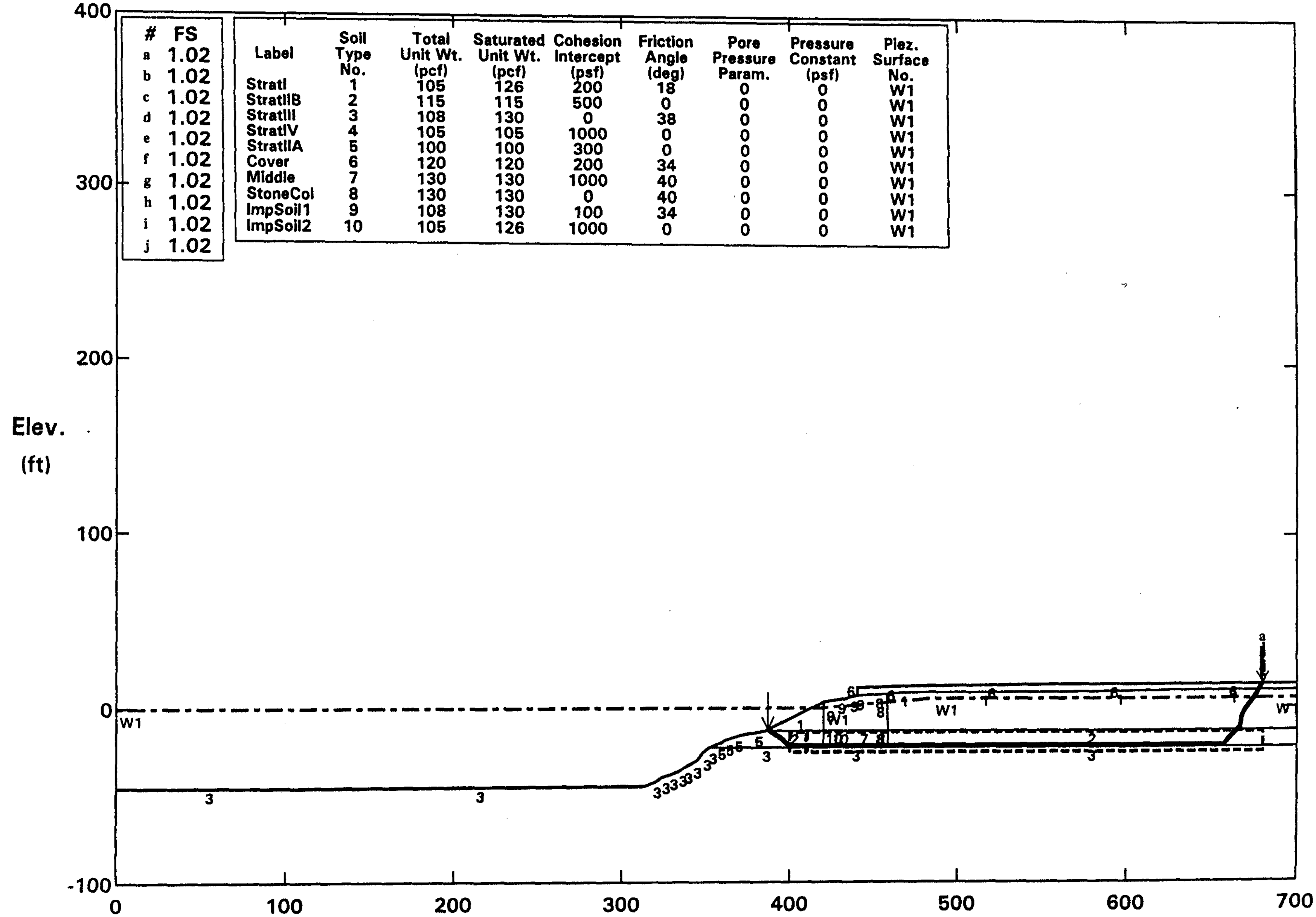
All surfaces evaluated. C:GGSCD-1.PLT By: P.T. 06-30-02 12:14pm



Factors Of Safety Calculated By The Modified Janbu Method

A-NAS - Section G-G', StoneCol w/Cover Dynamic Janbu Block Search, 0.15g

Ten Most Critical. C:GGSCD-1.PLT By: P.T. 06-30-02 12:14pm



#	FS
a	1.02
b	1.02
c	1.02
d	1.02
e	1.02
f	1.02
g	1.02
h	1.02
i	1.02
j	1.02

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	200	18	0	0	W1
StratIIB	2	115	115	500	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1000	0	0	0	W1
StratIIA	5	100	100	300	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
Middle	7	130	130	1000	40	0	0	W1
StoneCol	8	130	130	0	40	0	0	W1
ImpSoil1	9	108	130	100	34	0	0	W1
ImpSoil2	10	105	126	1000	0	0	0	W1

PCSTABL5M FSmin = 1.02 X-Axis (ft)

Factors Of Safety Ca d By The Modified Janbu Method

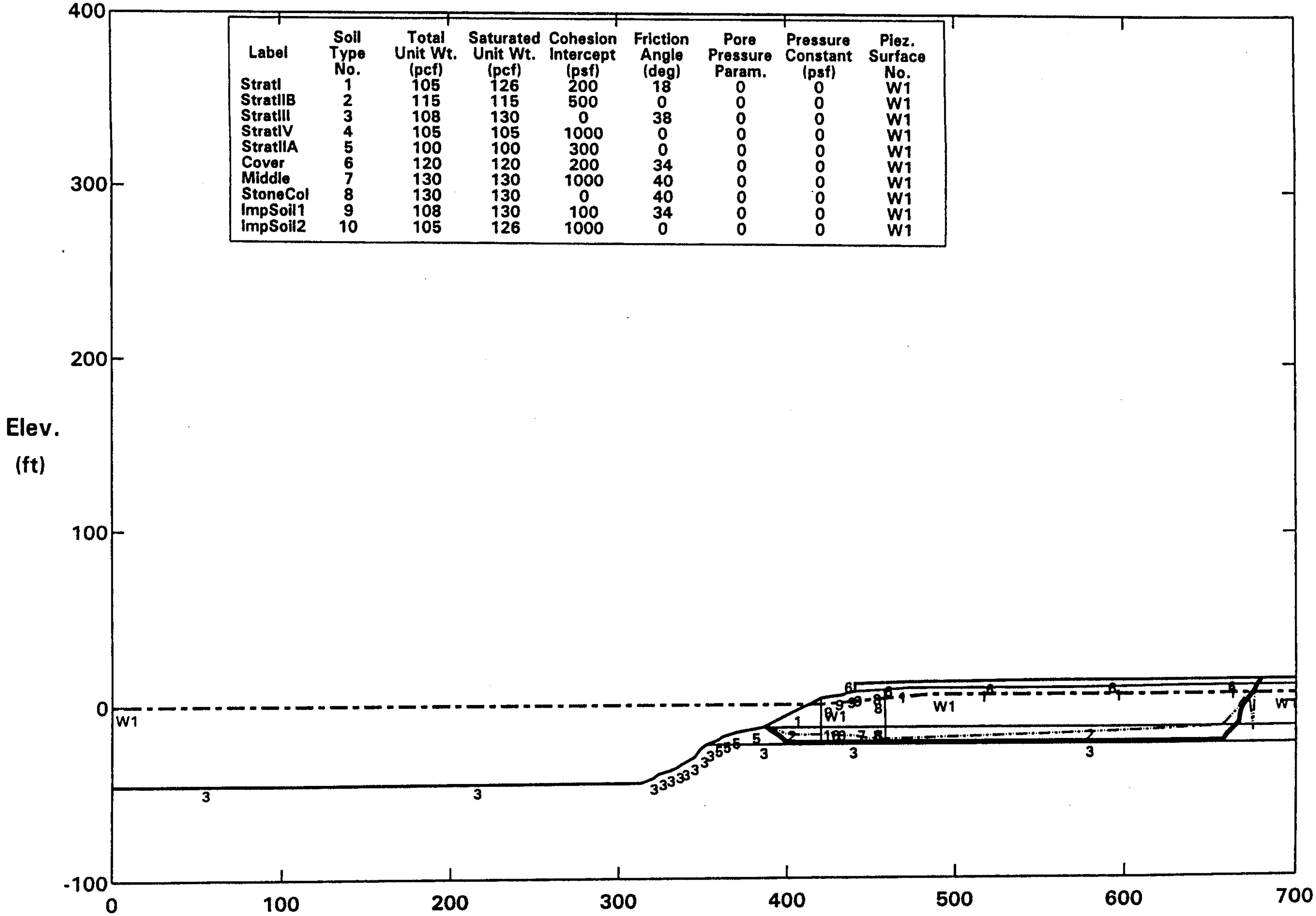
```
PROFIL       C:\GEO\STED\A-NAS\G-FINAL\GGSCD-1.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section G-G', StoneCol w/Cover  Dynamic Janbu Block Search, 0.15g
40 23
0. 54. 110. 54. 3
110. 54. 313. 55. 3
313. 55. 320. 58. 3
320. 58. 324. 60. 3
324. 60. 330. 62. 3
330. 62. 334. 64. 3
334. 64. 337. 66. 3
337. 66. 345. 70. 3
345. 70. 348. 74. 3
348. 74. 352. 77. 3
352. 77. 359. 80. 5
359. 80. 361. 81. 5
361. 81. 370. 84. 5
370. 84. 385. 87. 5
385. 87. 420. 103.1 1
420. 103.1 433. 105.1 9
433. 105.1 435. 106.1 9
435. 106.1 440. 107.1 9
440. 107.1 440.1 111.1 6
440.1 111.1 472. 112.1 6
472. 112.1 560. 112.7 6
560. 112.7 625. 113.1 6
625. 113.1 700. 113.1 6
419.9 87. 420. 103.1 9
440. 107.1 458. 107.7 8
458. 107.7 472. 108.1 1
458. 107.7 458.1 87. 8
472. 108.1 560. 108.7 1
560. 108.7 625. 109.1 1
625. 109.1 700. 109.1 1
385. 87. 420. 87. 2
419.8 77. 419.9 87. 10
419.9 87. 440. 87. 10
440. 87. 448. 87. 7
448. 87. 458.1 87. 8
458.1 87. 700. 87. 2
458.1 87. 458.2 77. 8
352. 77. 419.8 77. 3
419.8 77. 458.2 77. 3
458.2 77. 700. 77. 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
105. 126. 200. 18. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
```

```
WATER
1 62.4
4
0. 100.
420. 100.
485. 105.
700. 105.
EQUAKE
0.15 0. 0.
BLOCK -Sliding block, search
0
4000 2 5.
400. 80. 409.9 80. 12.
410. 80. 680. 80. 12.
```

A-NAS - Section G-G', StoneCol w/Cover Dynamic Spencer Block Search, 0.155g

Surface #1-GGSCD-1.OUT. C:GGSCD-1S.PLT By: P.T. 06-30-02 12:27pm

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	200	18	0	0	W1
StratIIB	2	115	115	500	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1000	0	0	0	W1
StratIIA	5	100	100	300	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
Middle	7	130	130	1000	40	0	0	W1
StoneCol	8	130	130	0	40	0	0	W1
ImpSoil1	9	108	130	100	34	0	0	W1
ImpSoil2	10	105	126	1000	0	0	0	W1



PCSTABL5M FS = 1.01 Theta = 1.12 X-Axis (ft)

Factors Of Safety Calculated By Spencer`s Method of Slices

```
PROFIL        C:\GEO\STED\A-NAS\G-FINAL\GGSCD-1S.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section G-G', StoneCol w/Cover  Dynamic Spencer Block Search, 0.155g
40 23
0. 54. 110. 54. 3
110. 54. 313. 55. 3
313. 55. 320. 58. 3
320. 58. 324. 60. 3
324. 60. 330. 62. 3
330. 62. 334. 64. 3
334. 64. 337. 66. 3
337. 66. 345. 70. 3
345. 70. 348. 74. 3
348. 74. 352. 77. 3
352. 77. 359. 80. 5
359. 80. 361. 81. 5
361. 81. 370. 84. 5
370. 84. 385. 87. 5
385. 87. 420. 103.1 1
420. 103.1 433. 105.1 9
433. 105.1 435. 106.1 9
435. 106.1 440. 107.1 9
440. 107.1 440.1 111.1 6
440.1 111.1 472. 112.1 6
472. 112.1 560. 112.7 6
560. 112.7 625. 113.1 6
625. 113.1 700. 113.1 6
419.9 87. 420. 103.1 9
440. 107.1 458. 107.7 8
458. 107.7 472. 108.1 1
458. 107.7 458.1 87. 8
472. 108.1 560. 108.7 1
560. 108.7 625. 109.1 1
625. 109.1 700. 109.1 1
385. 87. 420. 87. 2
419.8 77. 419.9 87. 10
419.9 87. 440. 87. 10
440. 87. 448. 87. 7
448. 87. 458.1 87. 8
458.1 87. 700. 87. 2
458.1 87. 458.2 77. 8
352. 77. 419.8 77. 3
419.8 77. 458.2 77. 3
458.2 77. 700. 77. 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Middle
StoneColImpSoil1ImpSoil2
10
105. 126. 200. 18. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 1000. 40. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
105. 126. 1000. 0. 0. 0. 1
```

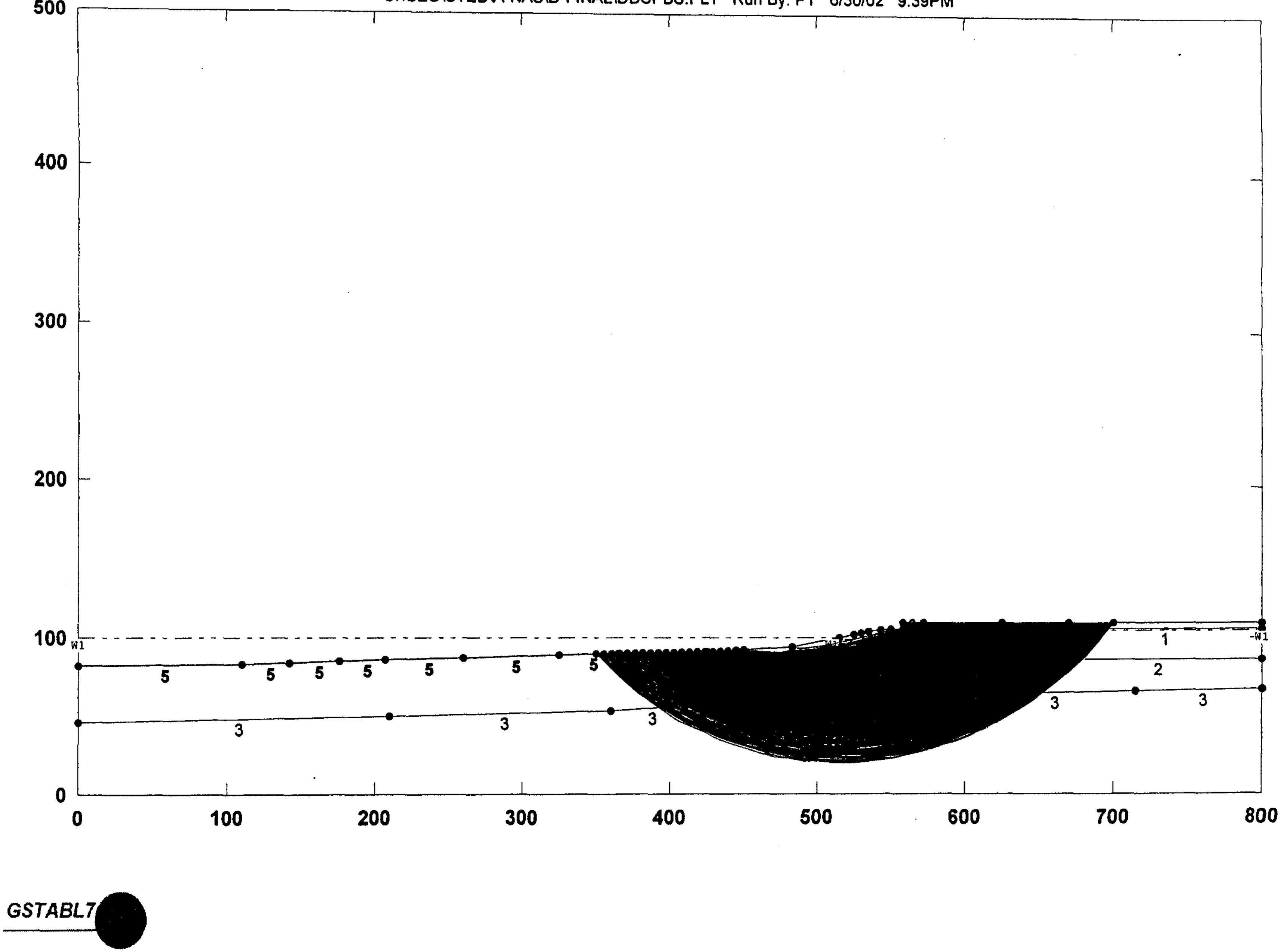
```
WATER
1 62.4
4
0. 100.
420. 100.
485. 105.
700. 105.
EQUAKE
0.155 0. 0.
SPENCR
5.
SURFAC #1-GGSCD-1.OUT
15
386.82 87.84
390.63 84.7
394.93 82.16
398.67 78.84
403.52 77.61
656.71 77.49
659.9 81.34
663.06 85.22
666.54 88.81
667.29 93.75
669.43 98.27
672.7 102.06
675.8 105.98
677.86 110.54
679.5 113.1
EXECUT
```

APPENDIX A3

ALTERNATIVE 3 – SHEET PILES WITH ANCHORS

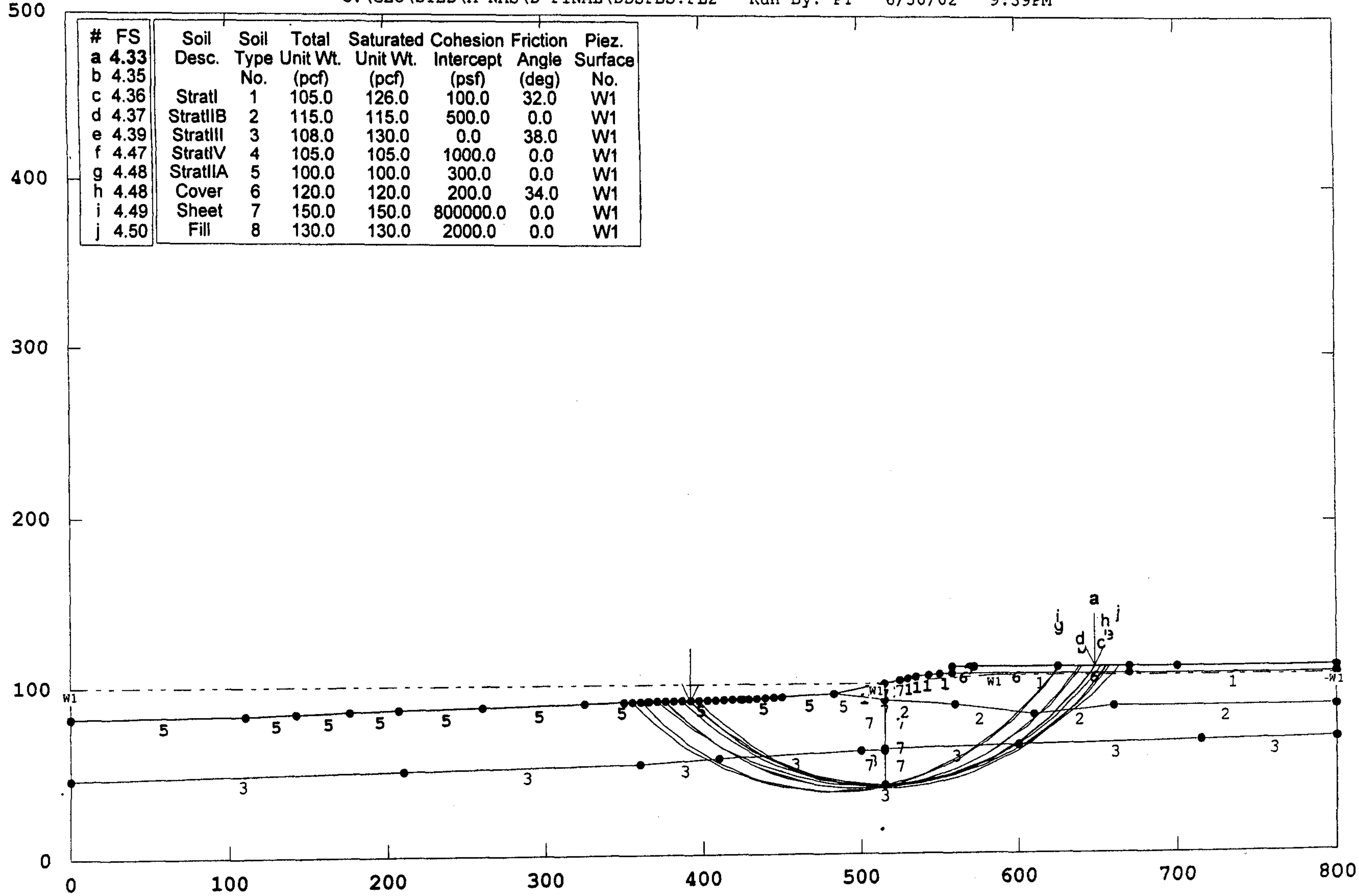
A-NAS - Section D-D', Sheet Piles Static Long-Term, Bishop Circular Search

C:\GEO\STED\A-NAS\D-FINAL\DDSPBS.PLT Run By: PT 6/30/02 9:39PM



A-NAS - Section D-D', Sheet Piles Static Long-Term, Bishop Circular Search

C:\GEO\STED\A-NAS\D-FINAL\DDSPBS.PL2 Run By: PT 6/30/02 9:39PM



GSTABL7 v.2 FSmin=4.33

Safety Factors Are Calculated By The Modified Bishop Method

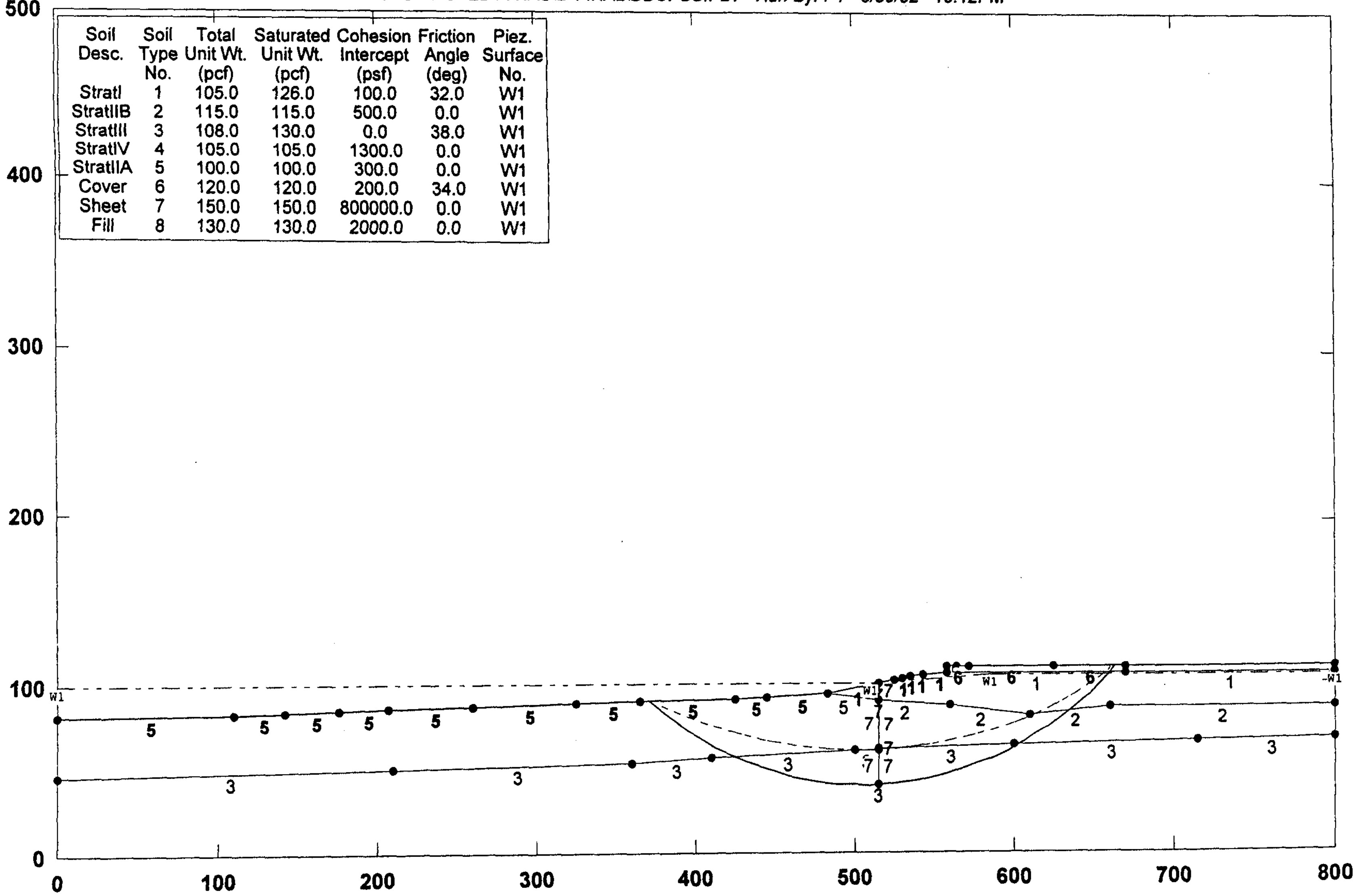
GSTABL7

```
PROFIL  C:\geo\sted\a-nas\d-final\ddspbs.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS - Section D-D', Sheet Piles       Static Long-Term, Bishop Circular Search
46 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 1
525. 102.1 530. 103.1 1
530. 103.1 535. 104.1 1
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 560. 87. 2
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
8
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
```

```
105. 105. 1000. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2
20 100
350. 450. 550. 700.
0. 10. 0. 0.
```

A-NAS - Section D-D', Sheet Piles Spencer Static Long-Term Slope Stability

C:\GEO\STED\A-NAS\D-FINAL\DDSPSS.PLT Run By: PT 6/30/02 10:12PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	100.0	32.0	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1300.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Sheet	7	150.0	150.0	800000.0	0.0	W1
Fill	8	130.0	130.0	2000.0	0.0	W1

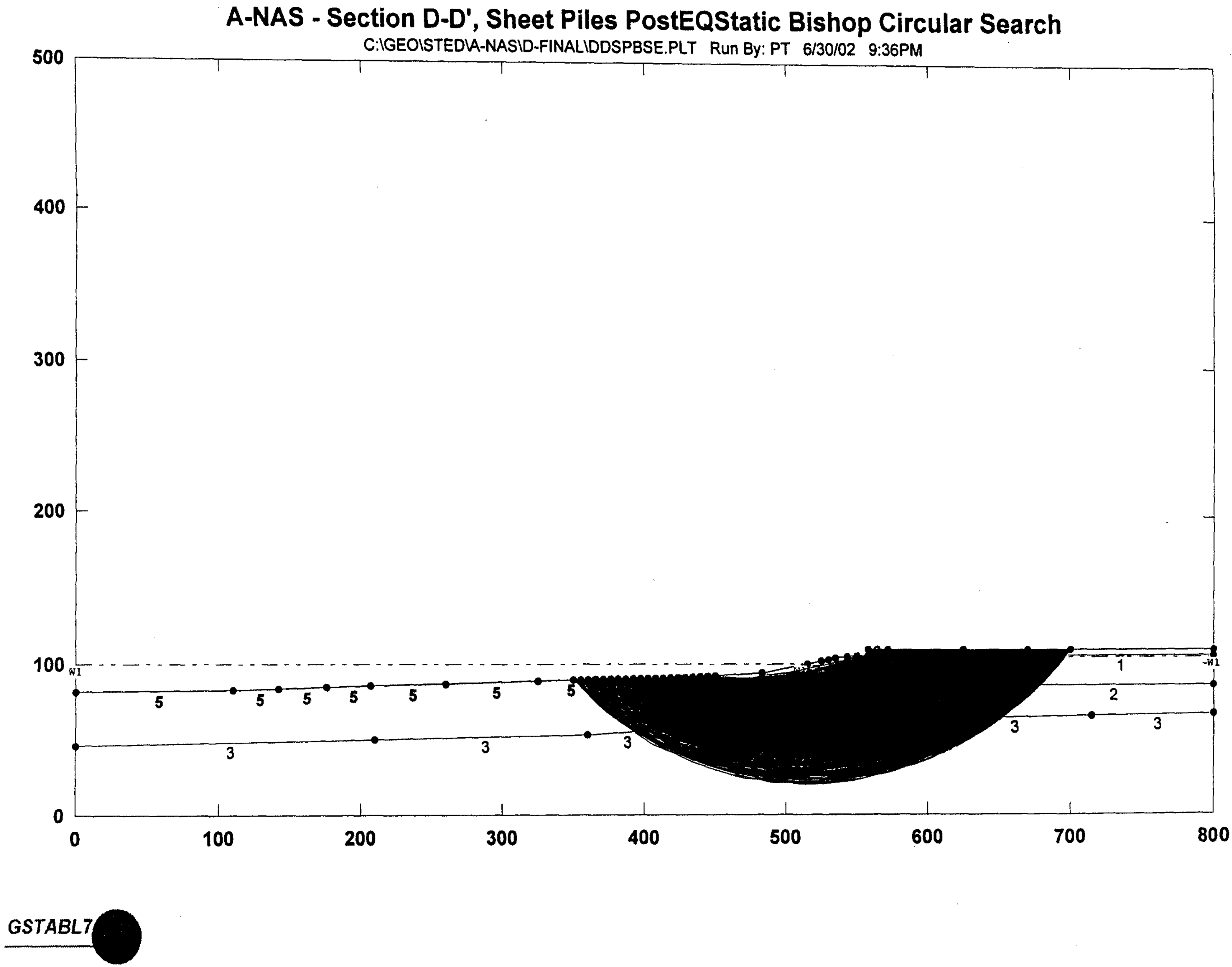
GSTABL7 v.2 FSmin=4.54

Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

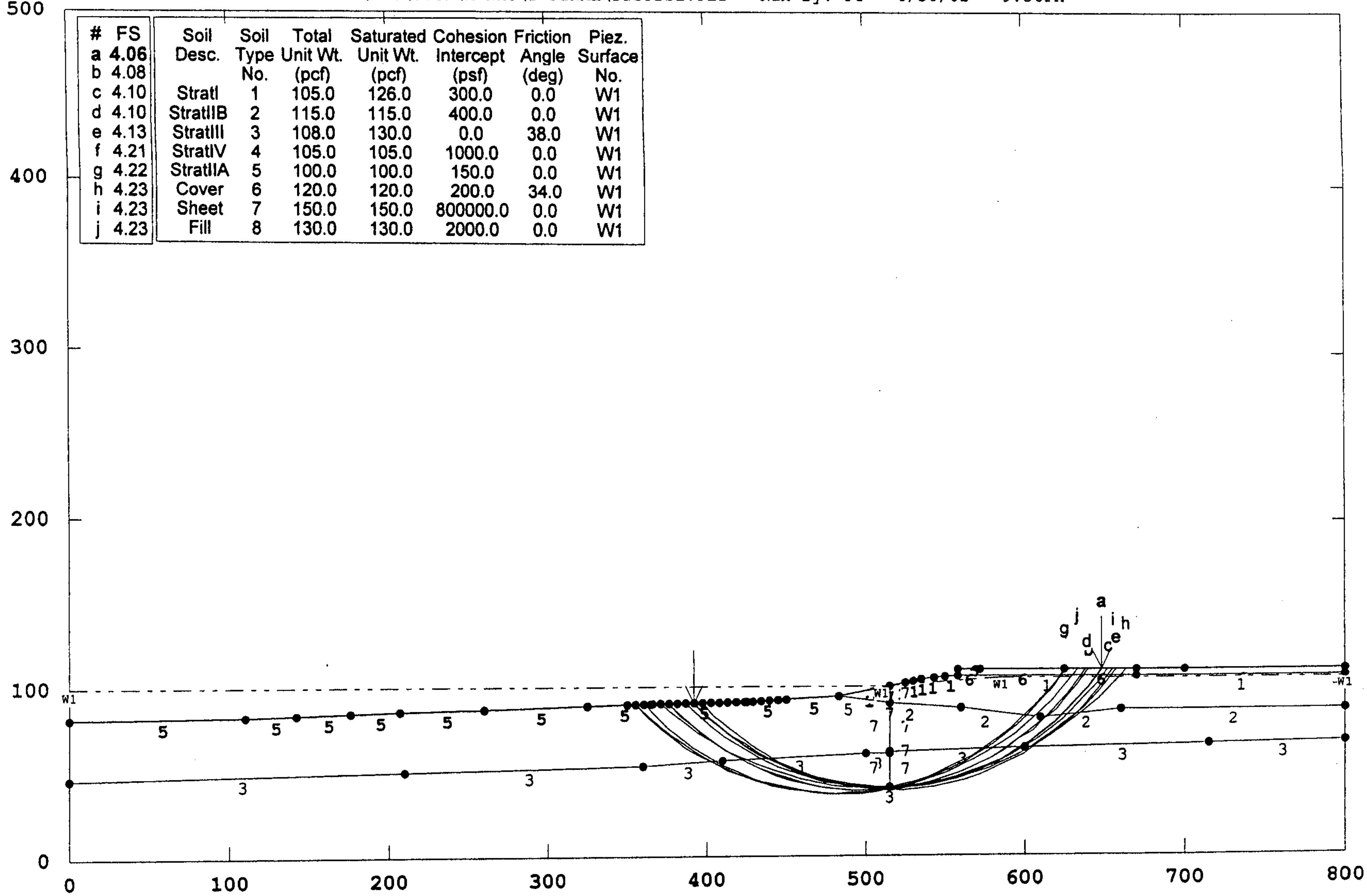
```
PROFIL  c:\geo\sted\a-nas\d-final\ddspss.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS - Section D-D', Sheet Piles        Spencer Static Long-Term Slope Stability
46 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 1
525. 102.1 530. 103.1 1
530. 103.1 535. 104.1 1
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 560. 87. 2
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
8
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
```

```
105. 105. 1300. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
GLEMS
4.
1      Water filled tension crack (0=no,1=yes)
0      Force Distribution (0=Single slice,1=Entire failure surf)
0      Select Method (0=Spencer,1=Morgenstern-Price)
2      ki function (Spencer=1 or 2, M-P=1, 2, 3, 4, or 5=user)
1.000  Lambda Coefficient (adjusts ki, 0.4 to 1.0)
0      Trial Lambda Adjustment option (0=no, 1=yes)
SURBIS
34
371.05 90.1
378.77 83.74
386.78 77.75
395.07 72.16
403.62 66.98
412.41 62.21
421.42 57.87
430.63 53.96
440.01 50.51
449.55 47.51
459.22 44.98
469.01 42.91
478.88 41.31
488.82 40.2
498.8 39.56
508.79 39.4
518.79 39.73
528.76 40.53
538.67 41.82
548.52 43.57
558.27 45.81
567.9 48.5
577.38 51.66
586.71 55.27
595.85 59.33
604.78 63.82
613.49 68.74
621.95 74.07
630.15 79.8
638.06 85.91
645.67 92.41
652.95 99.26
659.9 106.45
663.1 110.1
EXECUT
```


A-NAS - Section D-D', Sheet Piles PostEQStatic Bishop Circular Search
C:\GEO\STED\A-NAS\D-FINAL\DDSPBSE.PLT Run By: PT 6/30/02 9:36PM
500
400
300
200
100
0
0
100
200
300
400
500
600
700
800
W1
5
3
1
2
GSTABL7

A-NAS - Section D-D', Sheet Piles PostEQStatic Bishop Circular Search

C:\GEO\STED\A-NAS\D-FINAL\DDSPBSE.PL2 Run By: PT 6/30/02 9:36PM



#	FS
a	4.06
b	4.08
c	4.10
d	4.10
e	4.13
f	4.21
g	4.22
h	4.23
i	4.23
j	4.23

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	300.0	0.0	W1
StratIIB	2	115.0	115.0	400.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1000.0	0.0	W1
StratIIA	5	100.0	100.0	150.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Sheet	7	150.0	150.0	800000.0	0.0	W1
Fill	8	130.0	130.0	2000.0	0.0	W1

GSTABL7 v.2 FSmin=4.06

Safety Factors Are Calculated By The Modified Bishop Method

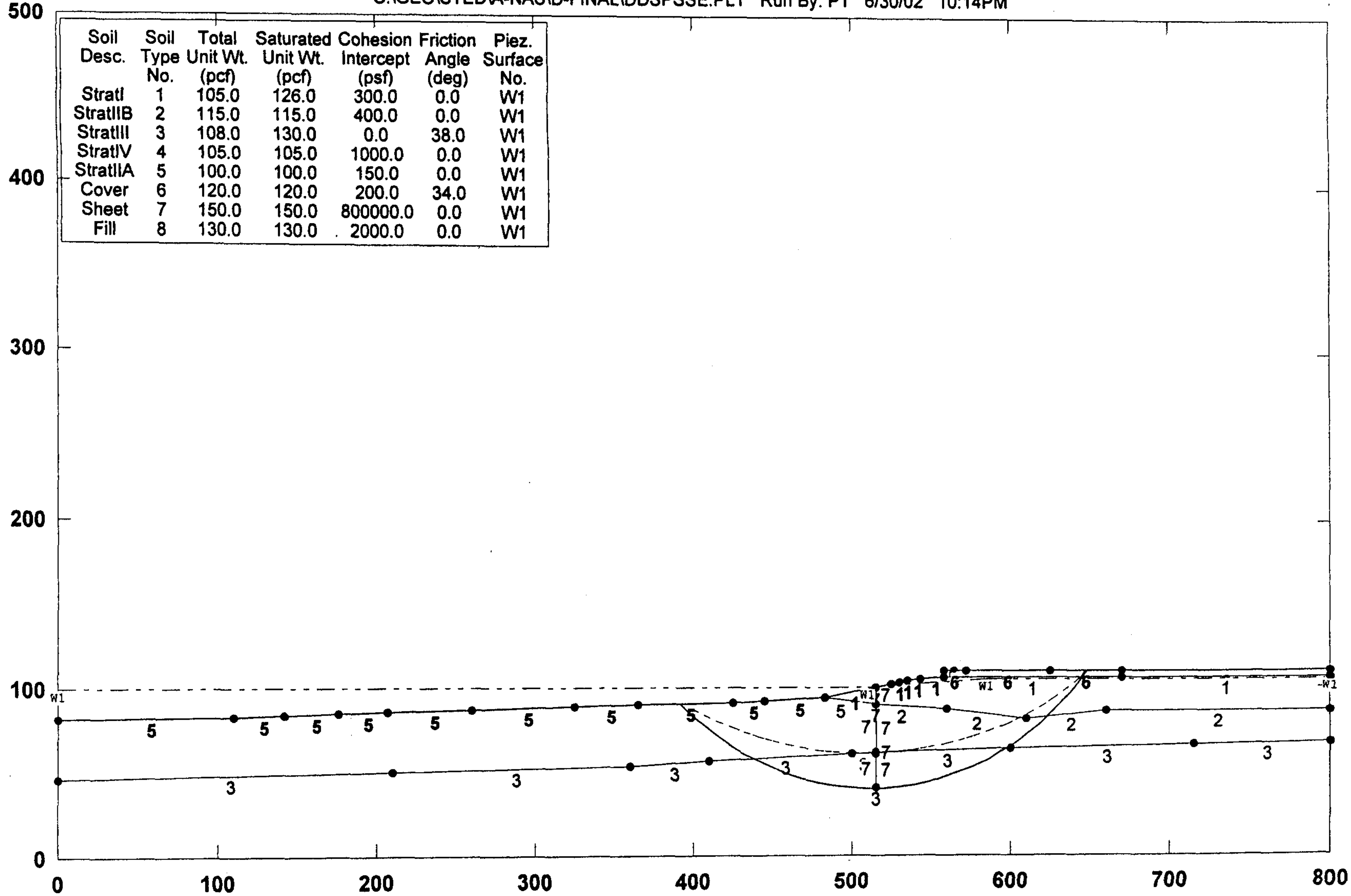
GSTABL7

```
PROFIL  c:\geo\sted\a-nas\d-final\ddspbse.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS - Section D-D', Sheet Piles PostEQStatic Bishop Circular Search
46 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 1
525. 102.1 530. 103.1 1
530. 103.1 535. 104.1 1
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 560. 87. 2
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
8
105. 126. 300. 0. 0. 0. 1
115. 115. 400. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
```

```
105. 105. 1000. 0. 0. 0. 1
100. 100. 150. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2
20 100
350. 450. 550. 700.
0. 10. 0. 0.
```

A-NAS - Section D-D', Sheet Piles PostEQSpencer Static Slope Stability

C:\GEO\STED\A-NAS\D-FINAL\DDSPSSE.PLT Run By: PT 6/30/02 10:14PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	300.0	0.0	W1
StratIIB	2	115.0	115.0	400.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1000.0	0.0	W1
StratIIA	5	100.0	100.0	150.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Sheet	7	150.0	150.0	800000.0	0.0	W1
Fill	8	130.0	130.0	2000.0	0.0	W1

GSTABL7 v.2 FSmin=4.13

Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-2)

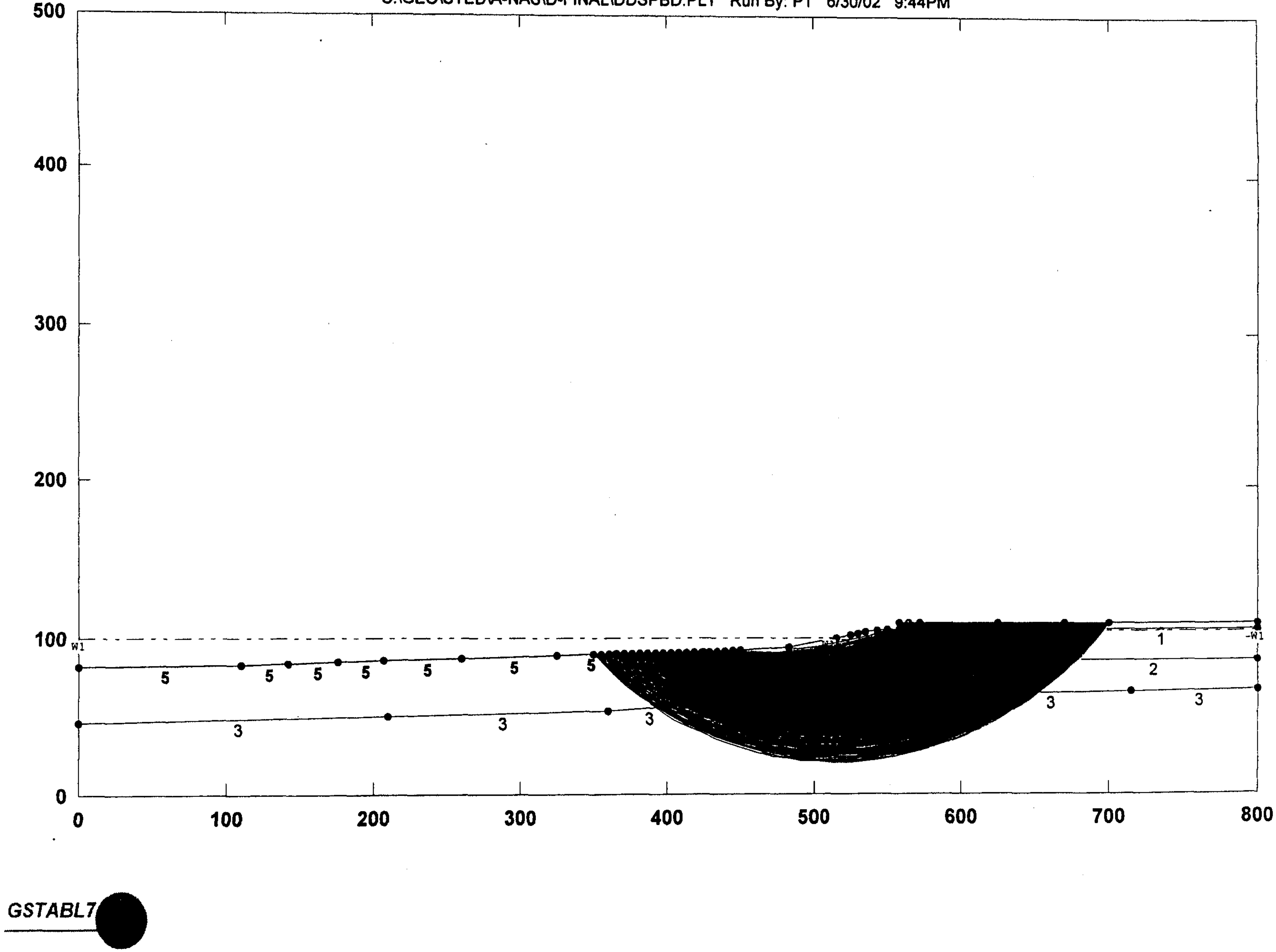
GSTABL7

```
PROFIL  c:\geo\sted\a-nas\d-final\ddspsse.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS - Section D-D', Sheet Piles PostEQSpencer Static Slope Stability
46 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 1
525. 102.1 530. 103.1 1
530. 103.1 535. 104.1 1
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 560. 87. 2
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
8
105. 126. 300. 0. 0. 0. 1
115. 115. 400. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
```

```
105. 105. 1000. 0. 0. 0. 1
100. 100. 150. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
GLEMS
4.
1       Water filled tension crack (0=no,1=yes)
0       Force Distribution (0=Single slice,1=Entire failure surf)
0       Select Method (0=Spencer,1=Morgenstern-Price)
2       ki function (Spencer=1 or 2, M-P=1, 2, 3, 4, or 5=user)
1.000   Lambda Coefficient (adjusts ki, 0.4 to 1.0)
0       Trial Lambda Adjustment option (0=no, 1=yes)
SURBIS
31
392.11 90.45
399.25 83.45
406.8 76.9
414.73 70.81
423.02 65.21
431.62 60.11
440.51 55.54
449.67 51.51
459.05 48.04
468.62 45.14
478.34 42.82
488.19 41.09
498.13 39.96
508.11 39.42
518.11 39.48
528.09 40.15
538.01 41.41
547.84 43.26
557.54 45.7
567.07 48.72
576.4 52.31
585.51 56.45
594.34 61.13
602.88 66.34
611.09 72.04
618.95 78.23
626.42 84.88
633.47 91.97
640.09 99.47
646.24 107.35
648.13 110.1
EXECUT
```

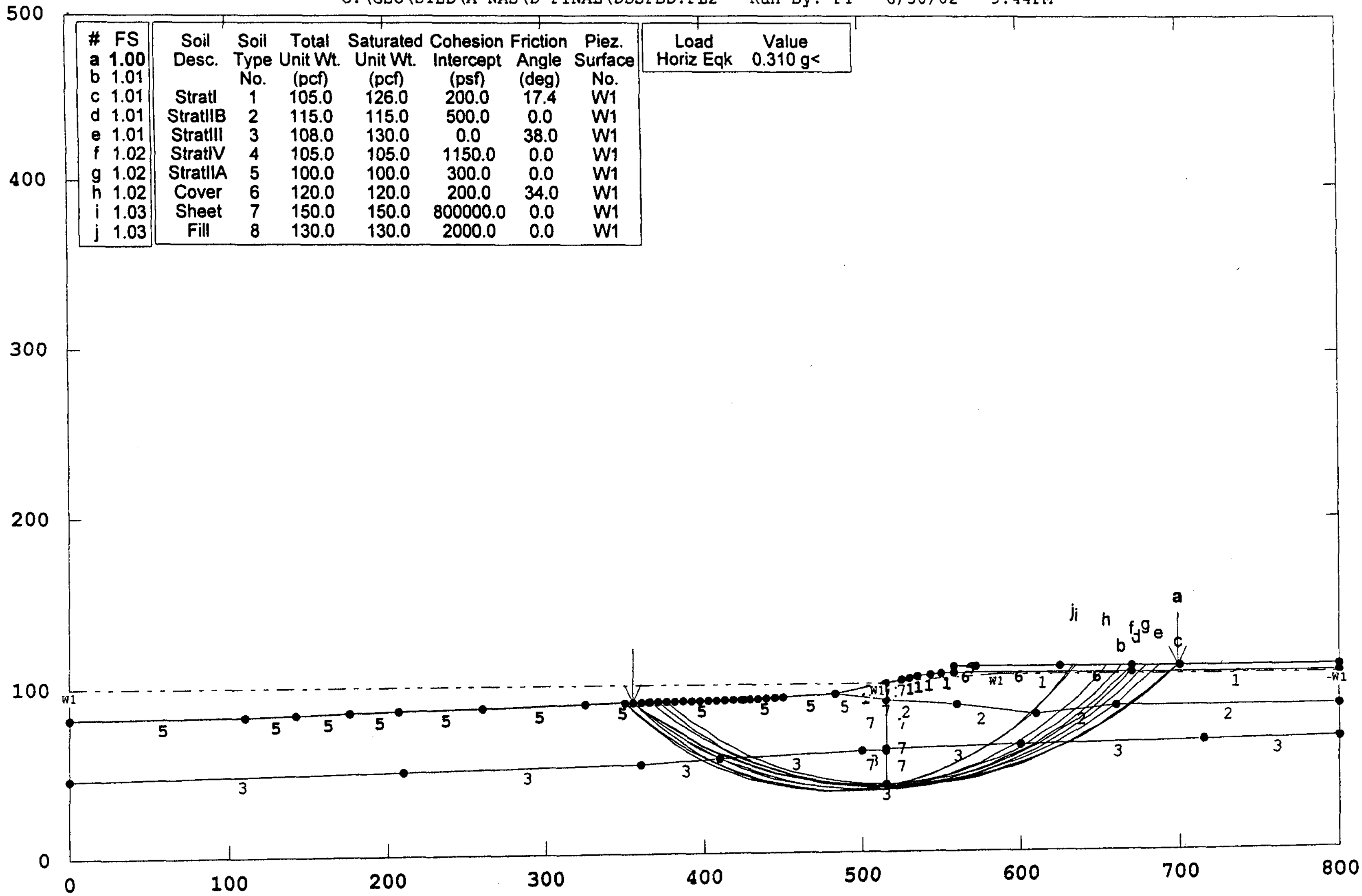
A-NAS - Section D-D', Sheet Piles EQ Dynamic Bishop Circular Search, 0.31g

C:\GEO\STED\A-NAS\D-FINAL\DDSPBD.PLT Run By: PT 6/30/02 9:44PM



A-NAS - Section D-D', Sheet Piles EQ Dynamic Bishop Circular Search, 0.31g

C:\GEO\STED\A-NAS\D-FINAL\DDSPBD.PL2 Run By: PT 6/30/02 9:44PM



#	FS
a	1.00
b	1.01
c	1.01
d	1.01
e	1.01
f	1.02
g	1.02
h	1.02
i	1.03
j	1.03

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	200.0	17.4	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1150.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Sheet	7	150.0	150.0	800000.0	0.0	W1
Fill	8	130.0	130.0	2000.0	0.0	W1

Load	Value
Horiz Eqk	0.310 g<

GSTABL7 v.2 FSmin=1.00

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

```
PROFIL  c:\geo\sted\a-nas\d-final\ddspbd.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS - Section D-D', Sheet Piles EQ    Dynamic Bishop Circular Search, 0.31g
46 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 1
525. 102.1 530. 103.1 1
530. 103.1 535. 104.1 1
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 560. 87. 2
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
8
105. 126. 200. 17.4 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
```

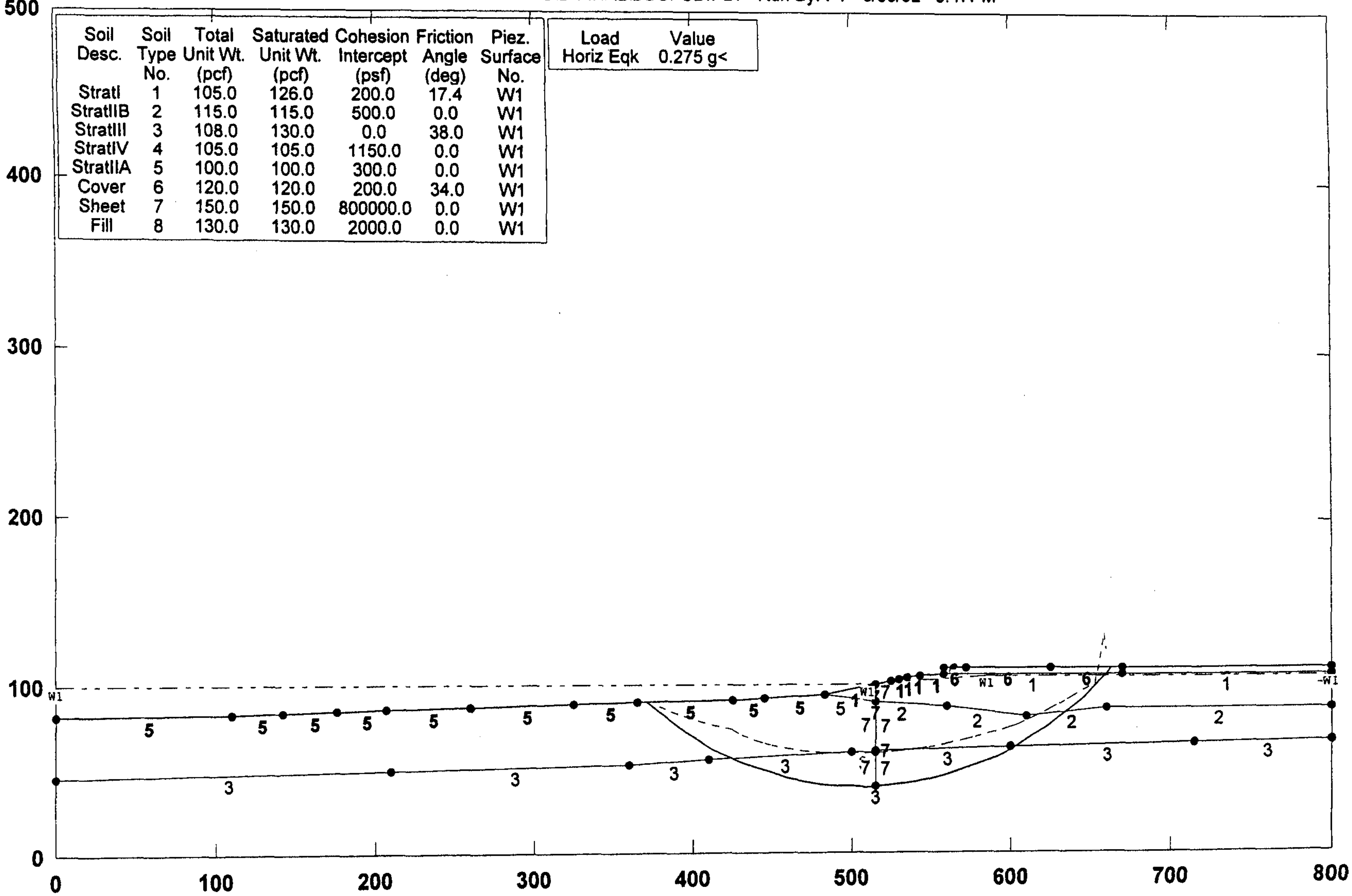
```
105. 105. 1150. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
EQUAKE
0.31 0. 0.
CIRCL2
20 100
350. 450. 550. 700.
0. 10. 0. 0.
```

A-NAS - Section D-D', Sheet Piles EQ Spencer Dynamic Slope Stability

C:\GEO\STED\A-NAS\D-FINAL\DDSPSD.PLT Run By: PT 6/30/02 9:47PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	200.0	17.4	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1150.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Sheet	7	150.0	150.0	800000.0	0.0	W1
Fill	8	130.0	130.0	2000.0	0.0	W1

Load	Value
Horiz Eqk	0.275 g<



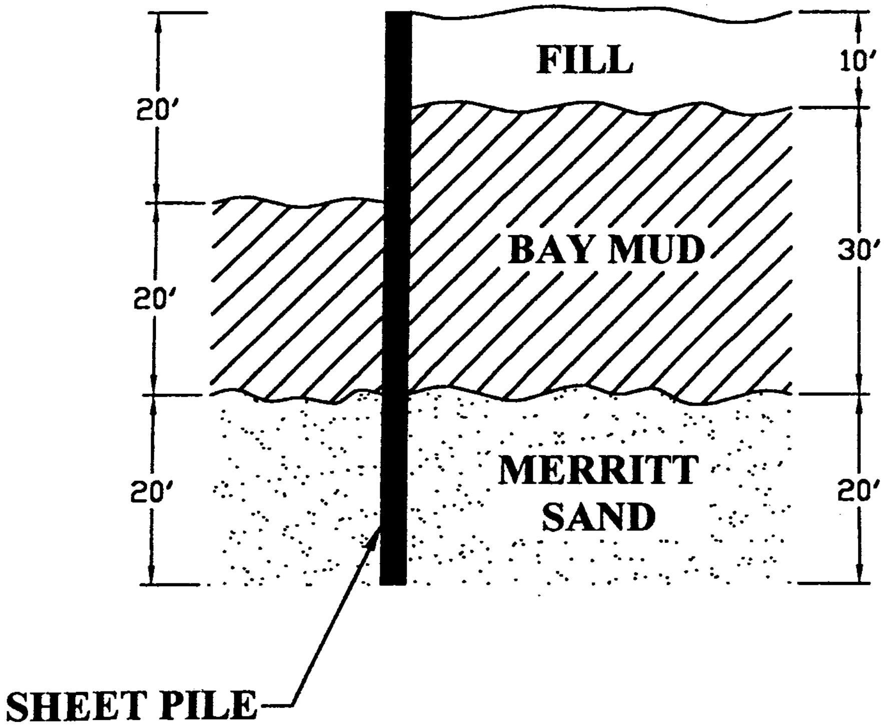
GSTABL7 v.2 FSmin=1.01

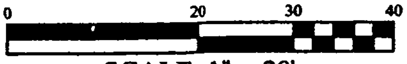
Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

```
PROFIL  c:\geo\sted\a-nas\d-final\ddspsd.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS - Section D-D', Sheet Piles EQ    Spencer Dynamic Slope Stability
46 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 1
525. 102.1 530. 103.1 1
530. 103.1 535. 104.1 1
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 560. 87. 2
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
8
105. 126. 200. 17.4 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
```

```
105. 105. 1150. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
EQUAKE
0.275 0. 0.
GLEMS
4.
1       Water filled tension crack (0=no,1=yes)
0       Force Distribution (0=Single slice,1=Entire failure surf)
0       Select Method (0=Spencer,1=Morgenstern-Price)
2       ki function (Spencer=1 or 2, M-P=1, 2, 3, 4, or 5=user)
1.000   Lambda Coefficient (adjusts ki, 0.4 to 1.0)
0       Trial Lambda Adjustment option (0=no, 1=yes)
SURBIS
34
371.05 90.1
378.77 83.74
386.78 77.75
395.07 72.16
403.62 66.98
412.41 62.21
421.42 57.87
430.63 53.96
440.01 50.51
449.55 47.51
459.22 44.98
469.01 42.91
478.88 41.31
488.82 40.2
498.8 39.56
508.79 39.4
518.79 39.73
528.76 40.53
538.67 41.82
548.52 43.57
558.27 45.81
567.9 48.5
577.38 51.66
586.71 55.27
595.85 59.33
604.78 63.82
613.49 68.74
621.95 74.07
630.15 79.8
638.06 85.91
645.67 92.41
652.95 99.26
659.9 106.45
663.1 110.1
EXECUT
```





SCALE: 1" = 20'

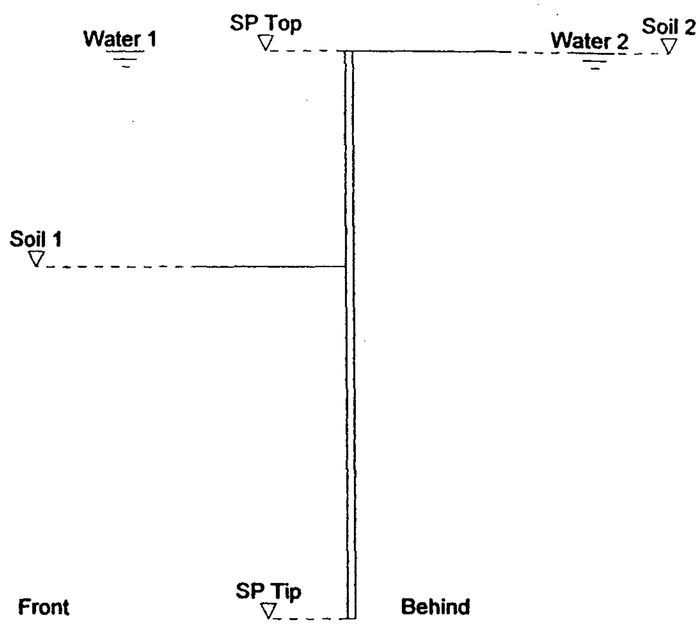
SECTION D-D'
CANTILIVERED SHEET PILE

SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/15/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\DStatic.spc
Comment:	Section D-D' Post EQ Soil Properties Cantilevered

GEODATA

Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	53.665
Soil Level in Front [ft]	20.000
Soil Level behind [ft]	0.000
Anchorlevel [ft]	0.000
Water Level in Front [ft]	0.000
Water Level behind [ft]	0.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	0.000
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Cantilever



LAYERS IN FRONT

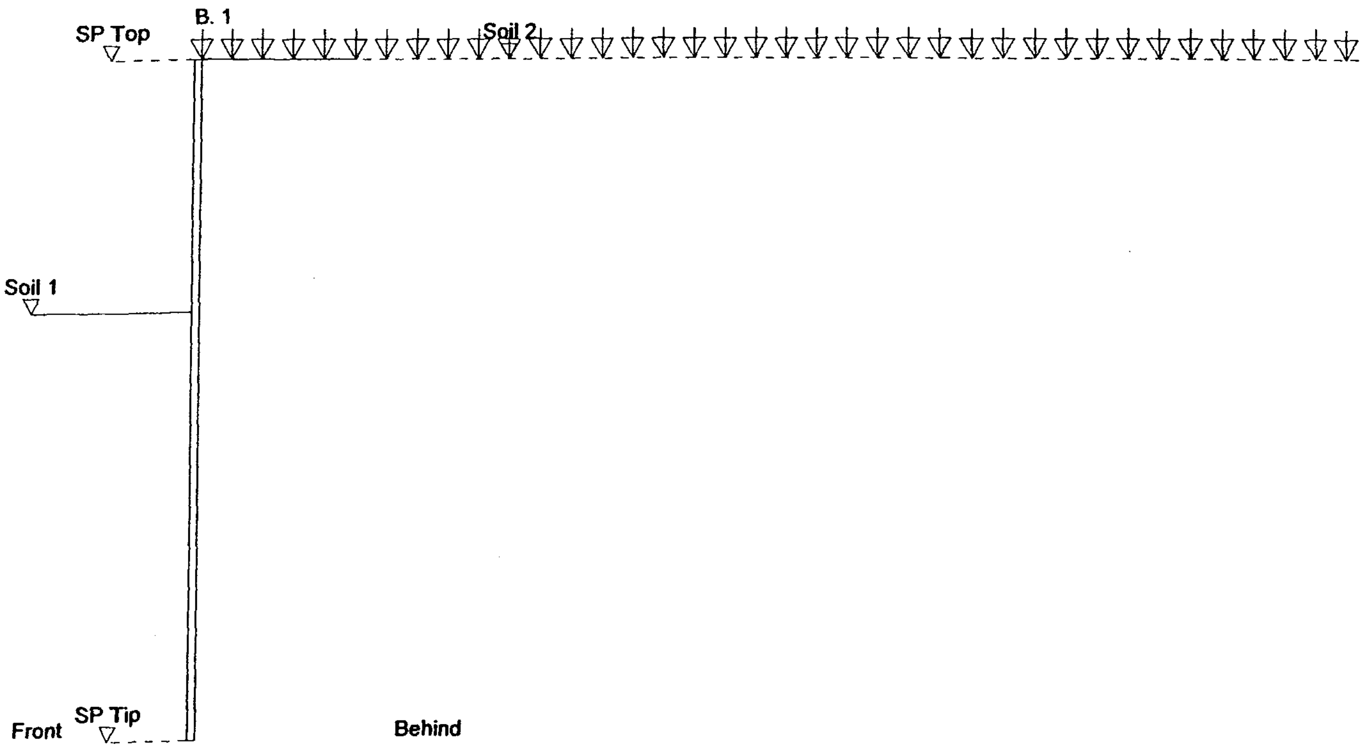
	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	40.000	0.115	0.052	1.000	0.000	0.000	0.400
Layer 2	120.000	0.130	0.067	4.208	38.000	0.000	0.000

LAYERS BEHIND

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kah	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	10.000	0.126	0.064	1.000	0.000	0.000	0.300
Layer 2	40.000	0.115	0.052	1.000	0.000	0.000	0.400
Layer 3	120.000	0.130	0.068	0.238	38.000	0.000	0.000

BOUSSINESQ

	Distance Wall [ft]	Width Surcharge [ft]	Depth Surcharge [ft]	Surcharge [kip/ft2]
Bousq. 1	0.000	300.000	0.000	0.480



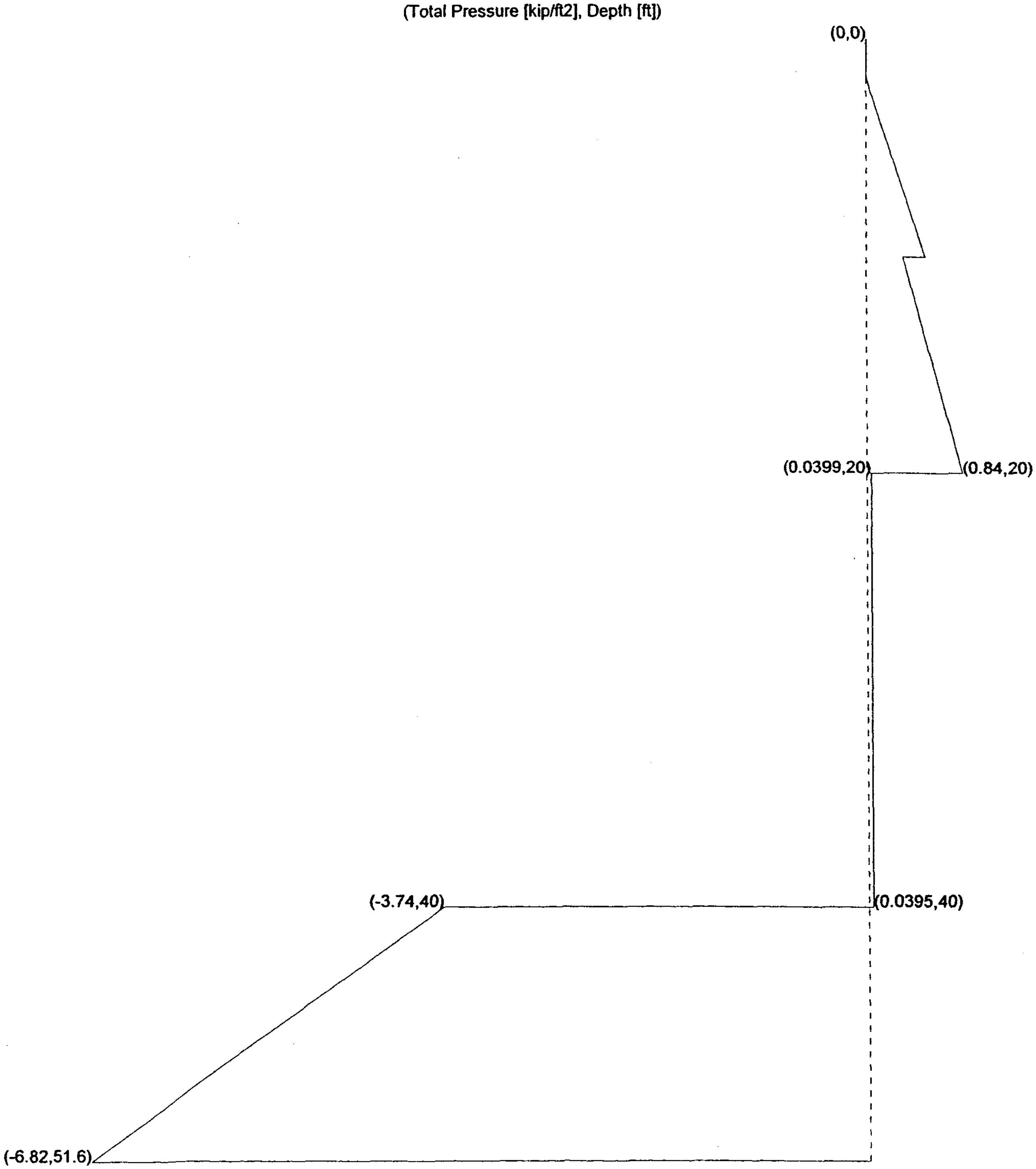
PILE SECTION

Name	AZ48
Inertia [in4/ft]	847.024
Modulus [in3/ft]	89.280
Area [in2/ft]	14.481
Mass [lbs/ft2]	49.279
Steelgrade [lb/in2]	60000.003
Requested Safety	2.000

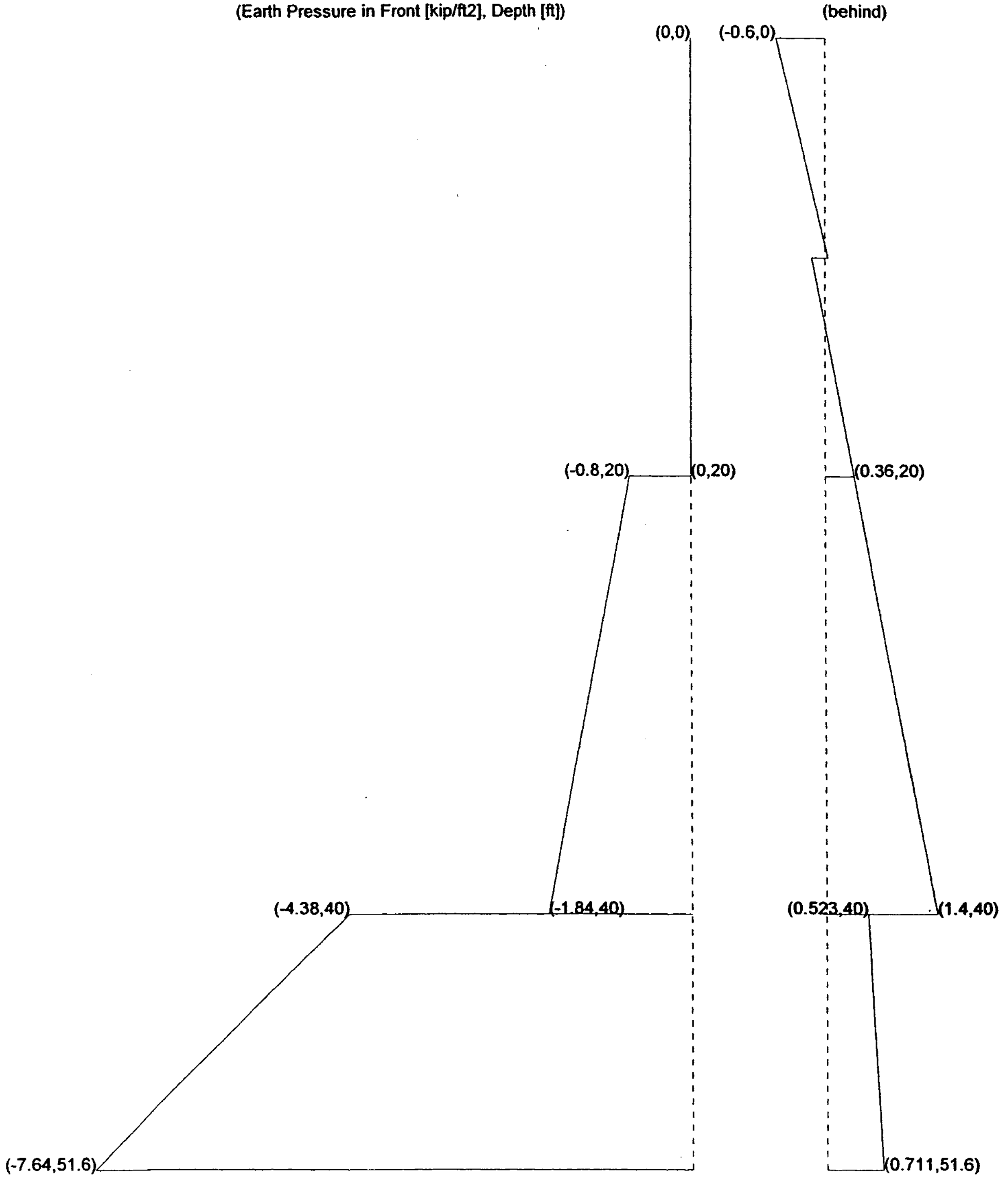
PILE CHECK

		Depth [ft]
Chosen Sheet Pile Section	AZ48	
Moment of Inertia [in4/ft]	847.024	
Section Modulus [in3/ft]	89.280	
Area [in2/ft]	14.481	
Mass [lbs/ft2]	49.279	
Steel Grade [lb/in2]	60000.003	
Minimal Moment [kipft/ft]	-2.387	51.612
Maximal Moment [kipft/ft]	227.526	42.174
Normal Forces at Min. Moment [kip/ft]	0.000	51.612
Normal Forces at Max. Moment [kip/ft]	0.000	42.174
Deflection at Min. Moment [ft]	0.000	51.612
Deflection at Max. Moment [ft]	-0.033	42.174
Min. Stress at Min. Moment [lb/in2]	-320.835	51.612
Max. Stress at Min. Moment [lb/in2]	320.835	51.612
Min. Stress at Max. Moment [lb/in2]	30580.264	42.174
Max. Stress at Max. Moment [lb/in2]	30580.264	42.174
Safety < Req. Safety = 2.000	1.962	
Pile Top [ft]		0.000
Pile Tip [ft]		53.665
Vertical Equilibrium [kip/ft]	0.000	
Anchor Force (horiz.) [kip/ft]	0.000	0.000

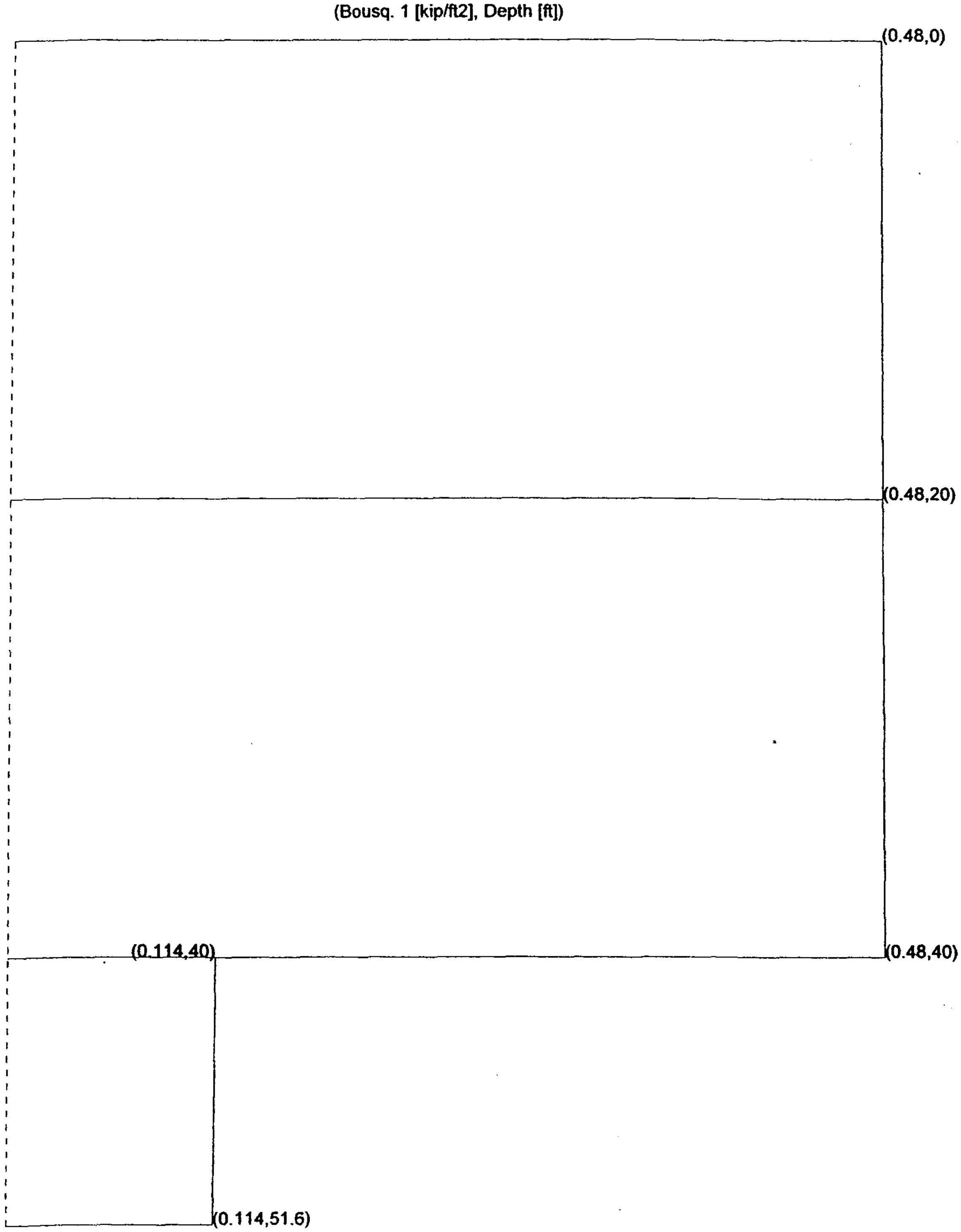
TOTAL PRESSURE DIAGRAM



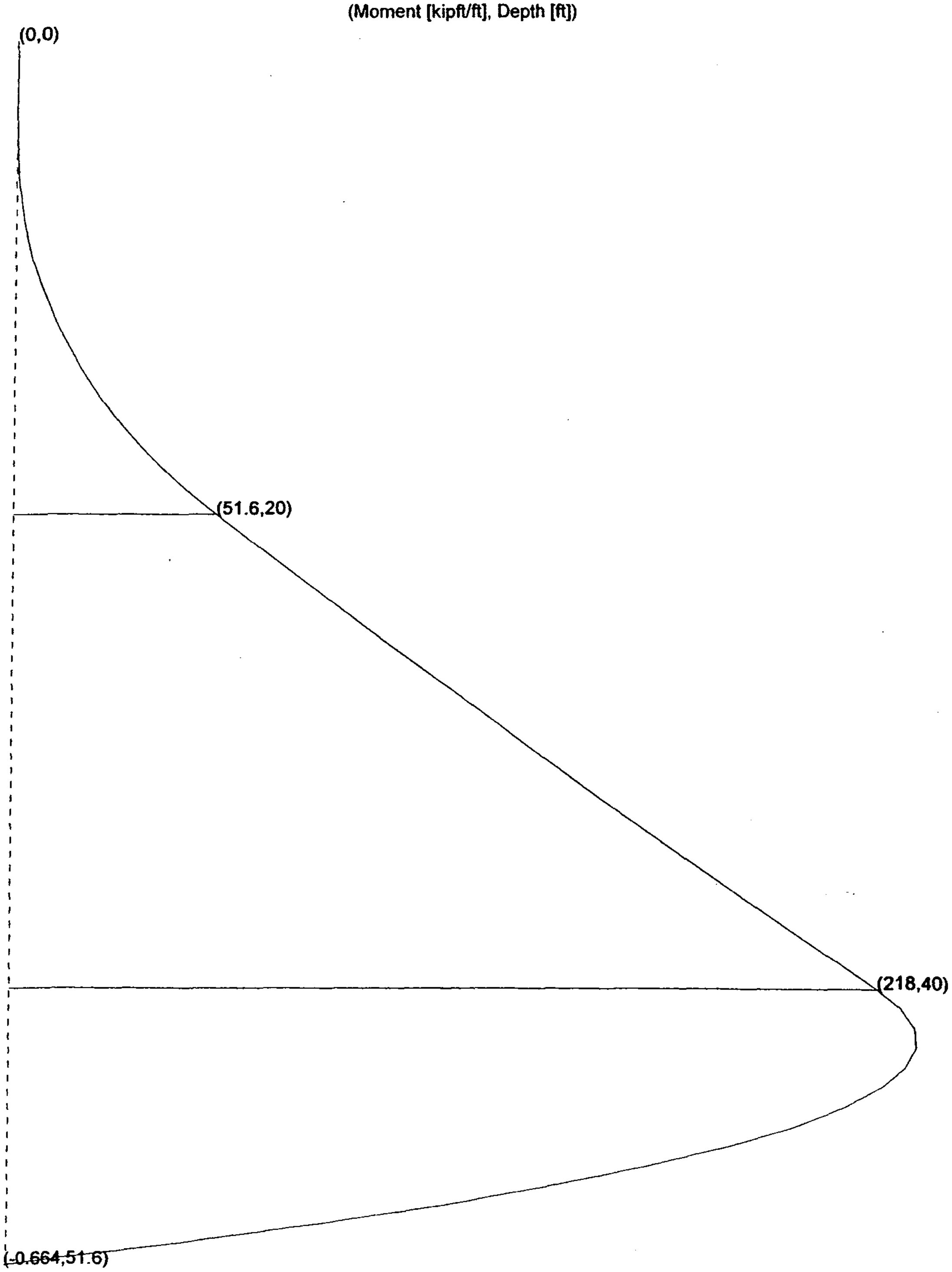
EARTH PRESSURE DIAGRAM



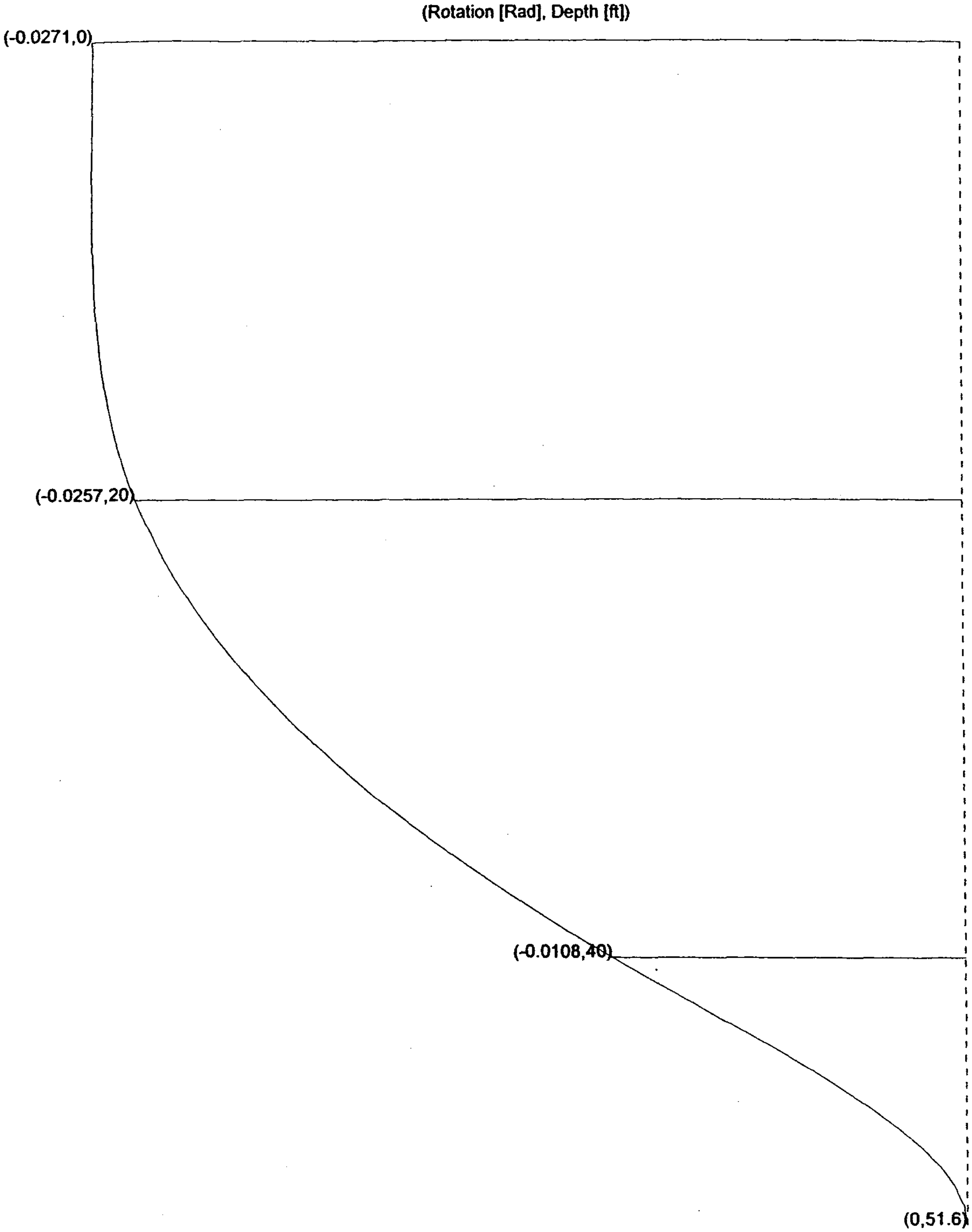
BOUSSINESQ DIAGRAM



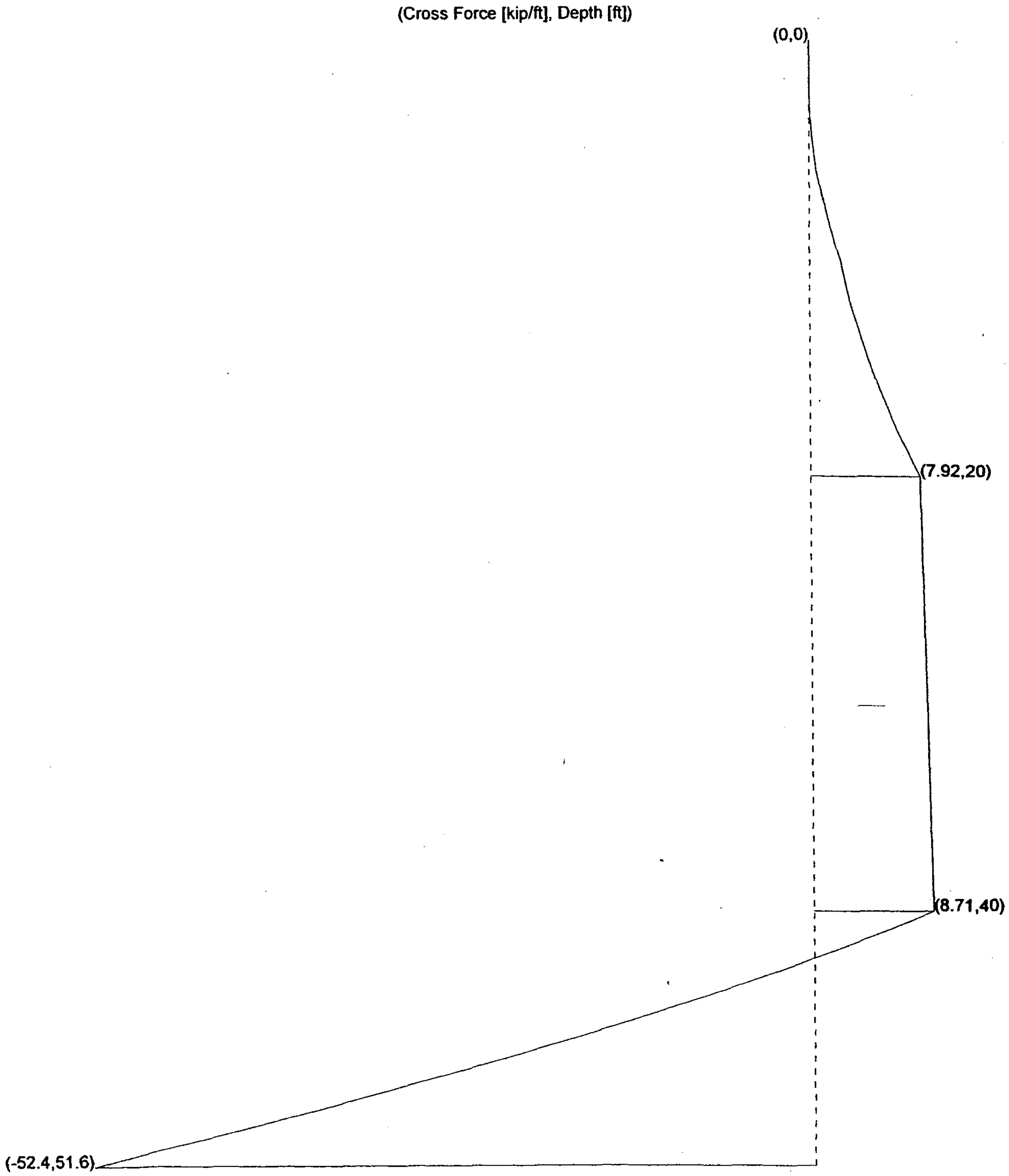
MOMENT DIAGRAM



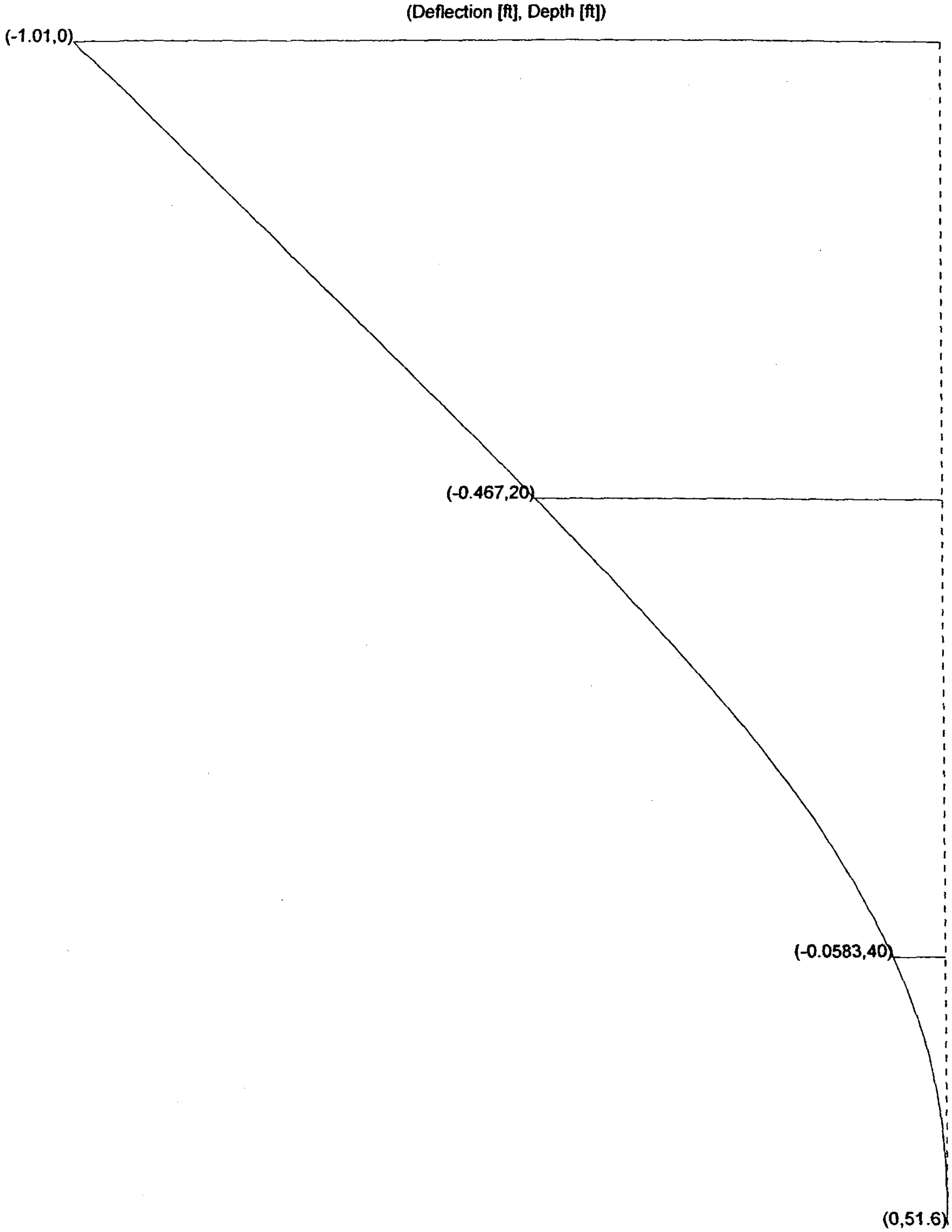
ROTATION DIAGRAM

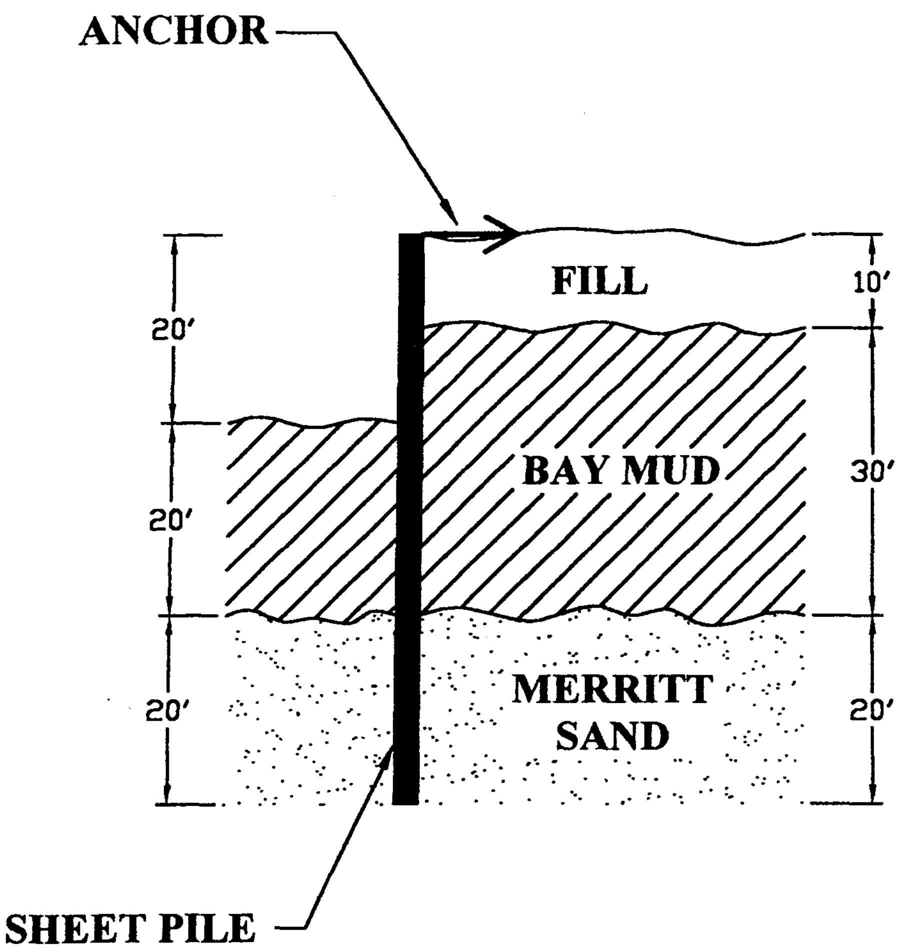


CROSS FORCE DIAGRAM



DEFLECTION DIAGRAM





SCALE: 1" = 20'

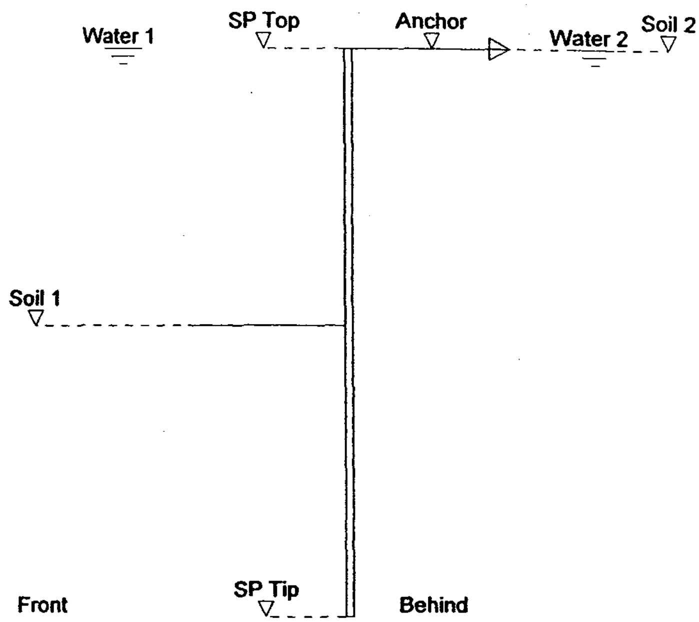
SECTION D-D'
ANCHORED SHEET PILE

SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/15/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\DStatic_a.spc
Comment:	Section D-D' Post EQ Soil Properties Anchored

GEODATA

Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	40.858
Soil Level in Front [ft]	20.000
Soil Level behind [ft]	0.000
Anchorlevel [ft]	0.000
Water Level in Front [ft]	0.000
Water Level behind [ft]	0.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	0.000
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Free



LAYERS IN FRONT

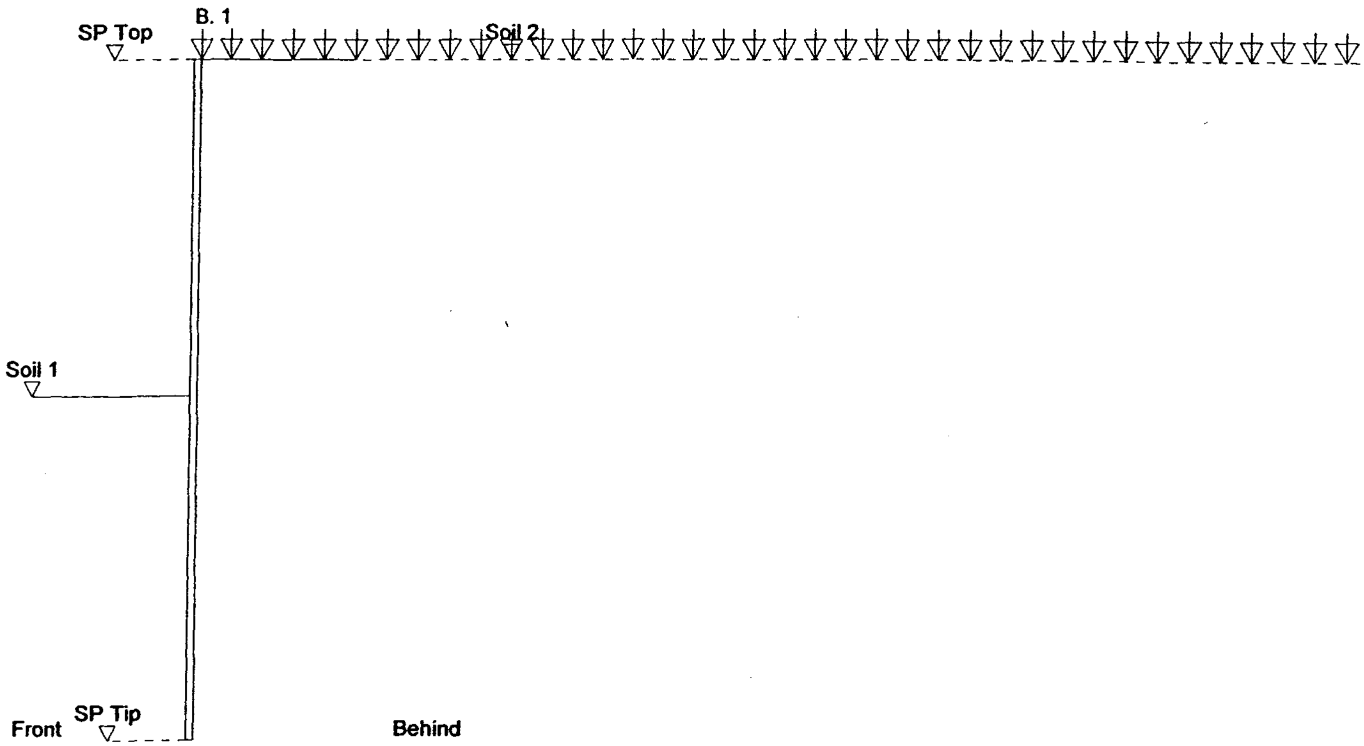
	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	40.000	0.115	0.052	1.000	0.000	0.000	0.400
Layer 2	120.000	0.130	0.067	4.208	38.000	0.000	0.000

LAYERS BEHIND

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kah	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	10.000	0.126	0.064	1.000	0.000	0.000	0.300
Layer 2	40.000	0.115	0.052	1.000	0.000	0.000	0.400
Layer 3	120.000	0.130	0.068	0.238	38.000	0.000	0.000

BOUSSINESQ

	Distance Wall [ft]	Width Surcharge [ft]	Depth Surcharge [ft]	Surcharge [kip/ft2]
Bousq. 1	0.000	300.000	0.000	0.480



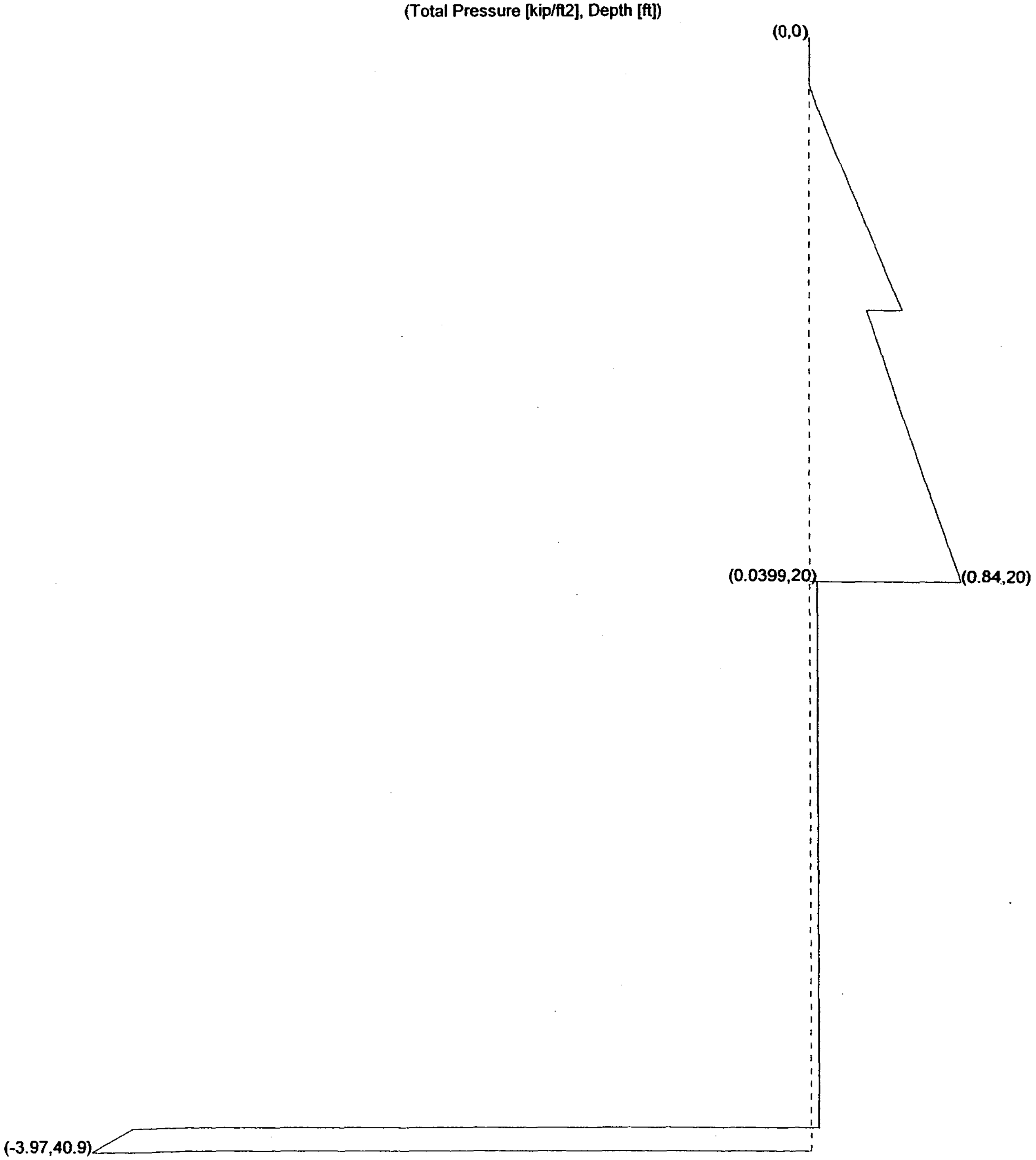
PILE SECTION

Name	AZ48
Inertia [in4/ft]	847.024
Modulus [in3/ft]	89.280
Area [in2/ft]	14.481
Mass [lbs/ft2]	49.279
Steelgrade [lb/in2]	60000.003
Requested Safety	2.000

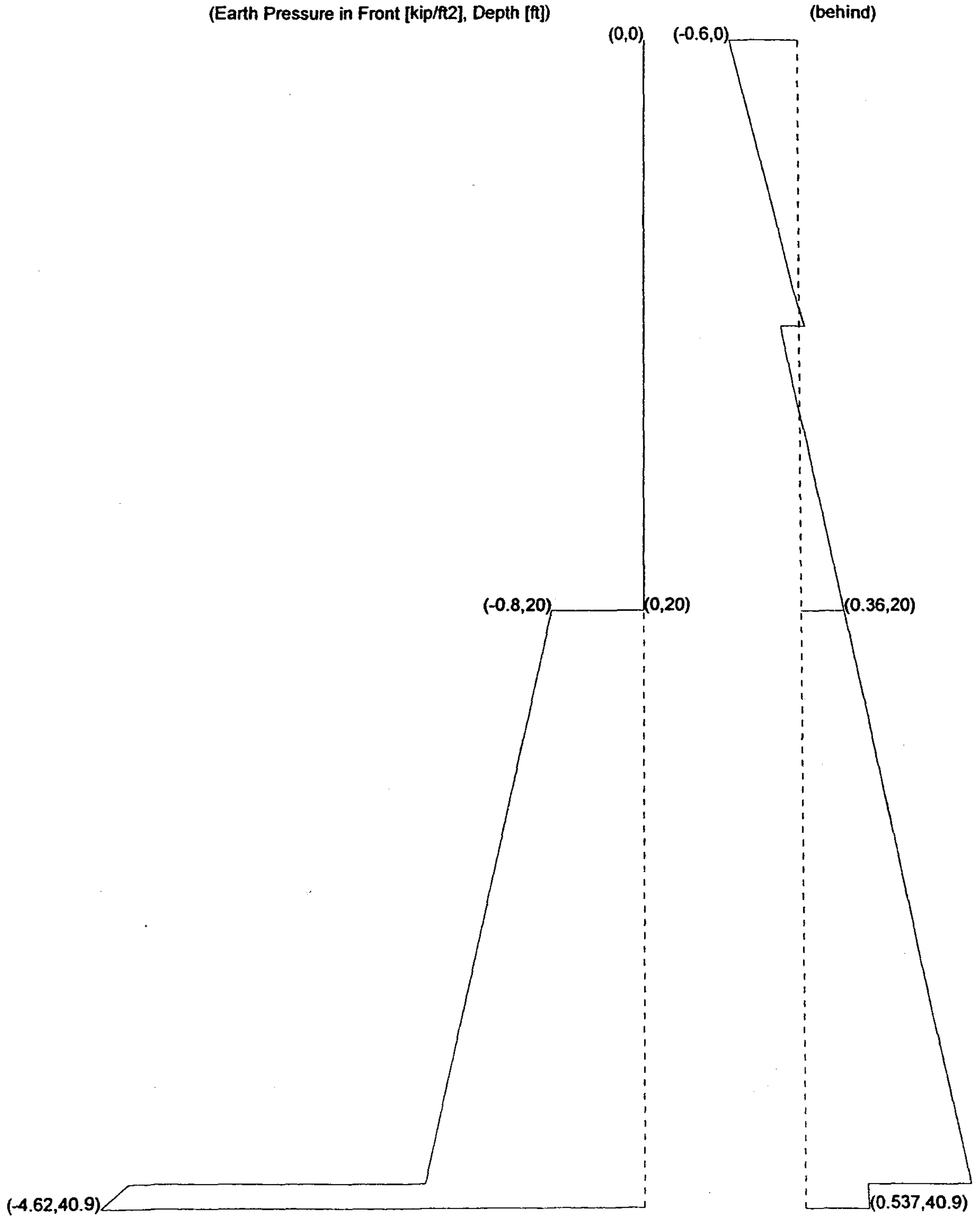
PILE CHECK

		Depth [ft]
Chosen Sheet Pile Section	AZ48	
Moment of Inertia [in4/ft]	847.024	
Section Modulus [in3/ft]	89.280	
Area [in2/ft]	14.481	
Mass [lbs/ft2]	49.279	
Steel Grade [lb/in2]	60000.003	
Minimal Moment [kipft/ft]	-61.810	16.686
Maximal Moment [kipft/ft]	0.000	40.853
Normal Forces at Min. Moment [kip/ft]	0.000	16.686
Normal Forces at Max. Moment [kip/ft]	0.000	40.853
Deflection at Min. Moment [ft]	-0.053	16.686
Deflection at Max. Moment [ft]	0.000	40.853
Min. Stress at Min. Moment [lb/in2]	-8307.448	16.686
Max. Stress at Min. Moment [lb/in2]	8307.448	16.686
Min. Stress at Max. Moment [lb/in2]	-0.048	40.853
Max. Stress at Max. Moment [lb/in2]	0.048	40.853
Safety > Req. Safety = 2.000	7.222	
Pile Top [ft]		0.000
Pile Tip [ft]		40.858
Vertical Equilibrium [kip/ft]	0.000	
Anchor Force (horiz.) [kip/ft]	-5.481	0.000

TOTAL PRESSURE DIAGRAM

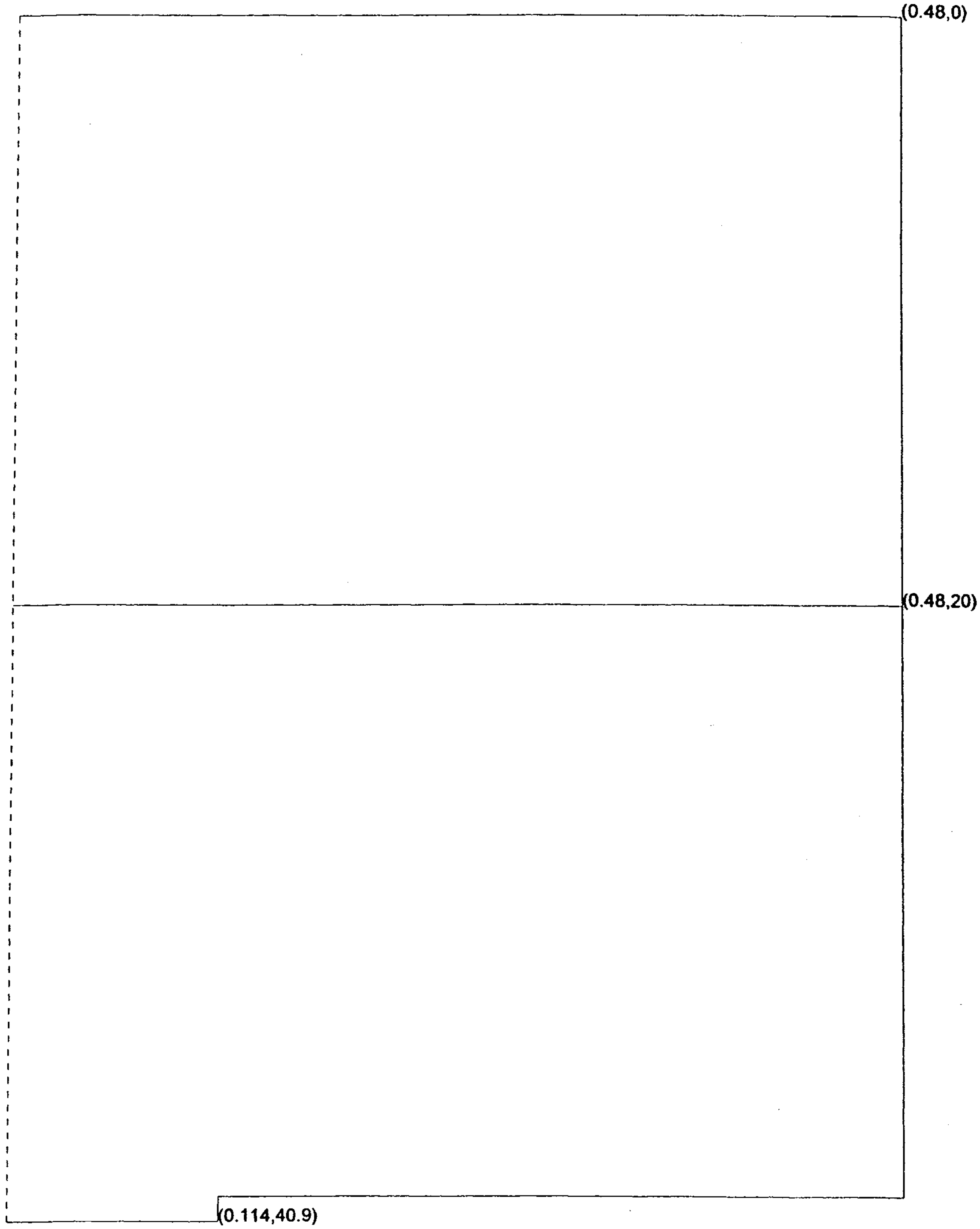


EARTH PRESSURE DIAGRAM

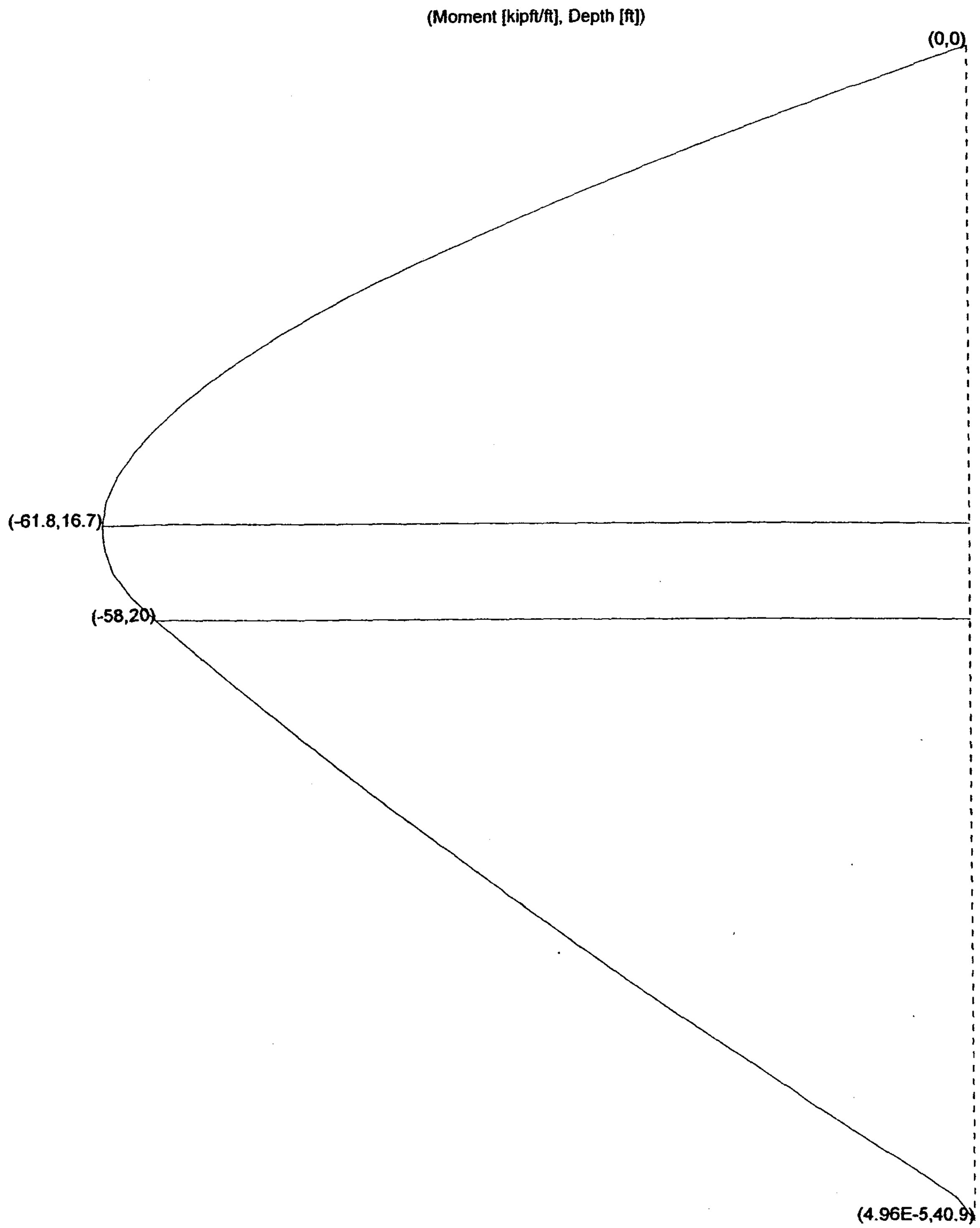


BOUSSINESQ DIAGRAM

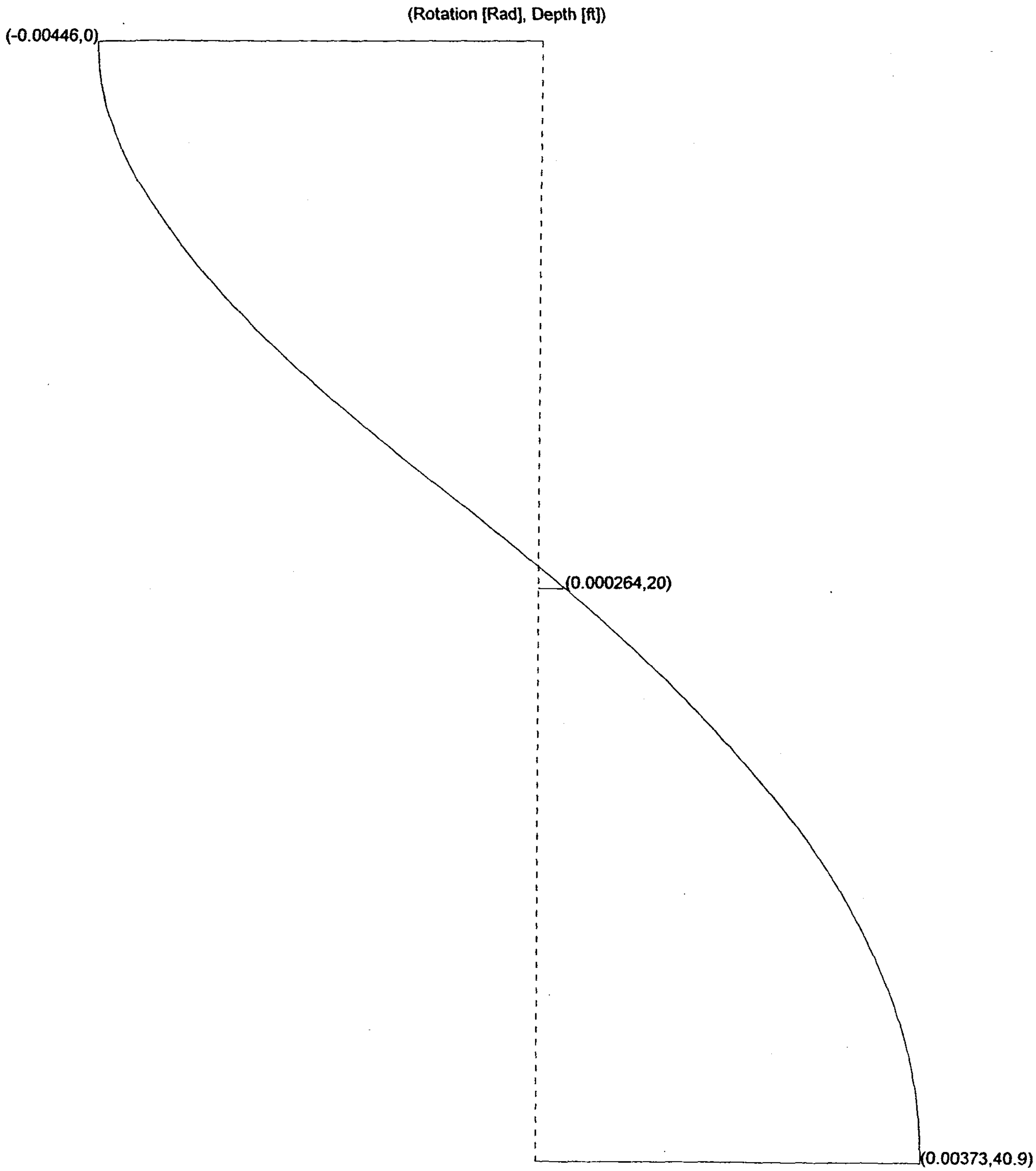
(Bousq. 1 [kip/ft2], Depth [ft])



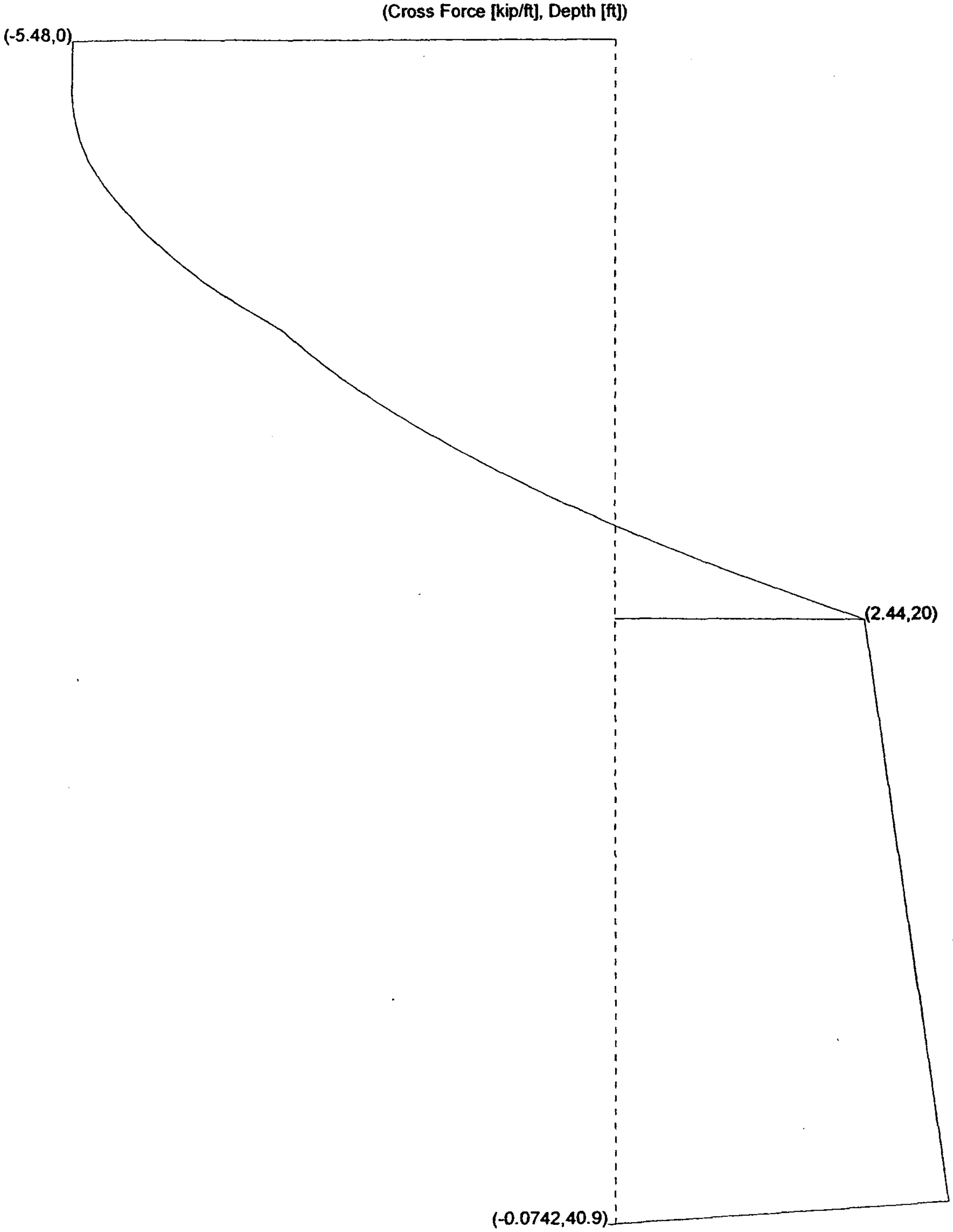
MOMENT DIAGRAM



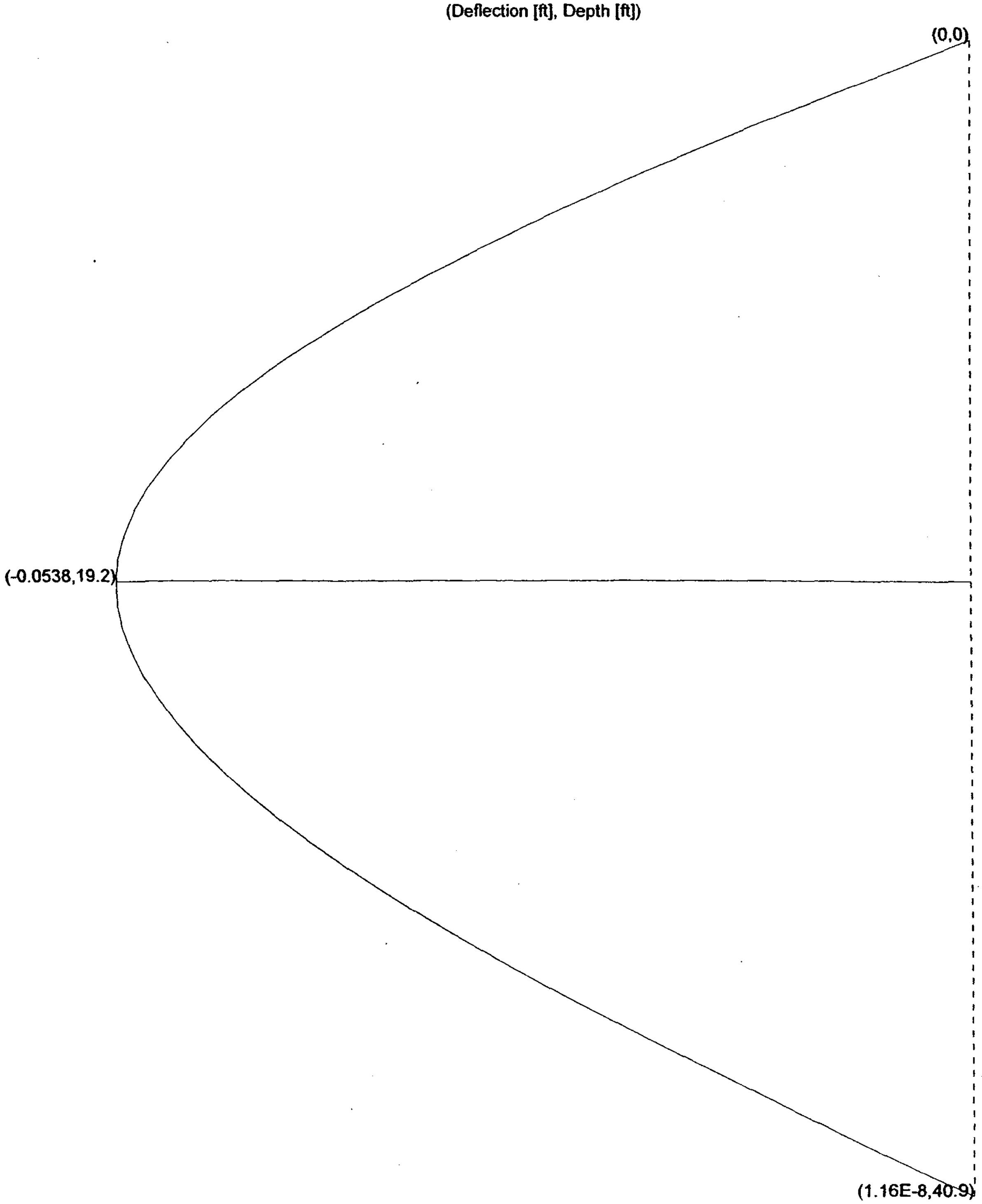
ROTATION DIAGRAM

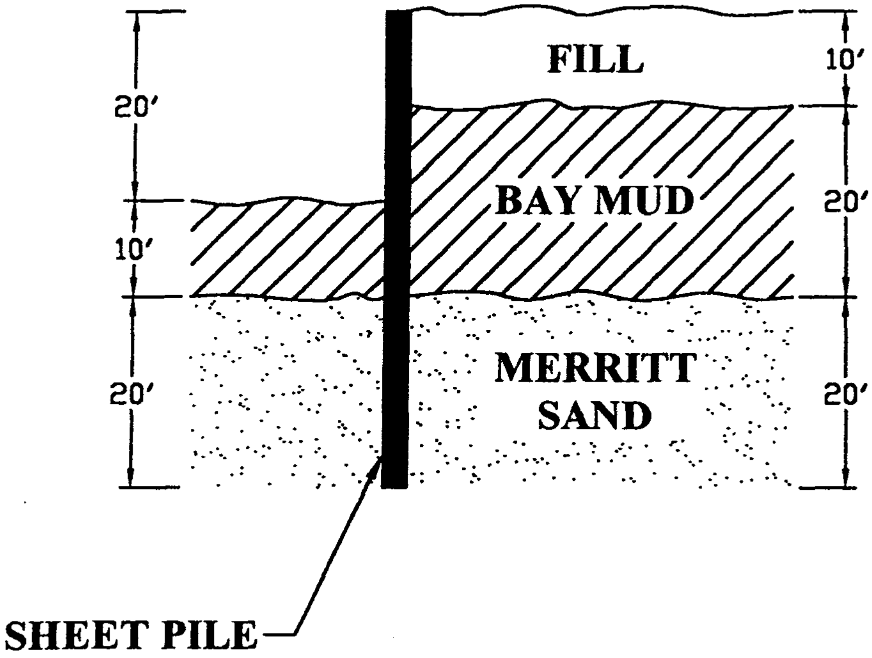


CROSS FORCE DIAGRAM



DEFLECTION DIAGRAM





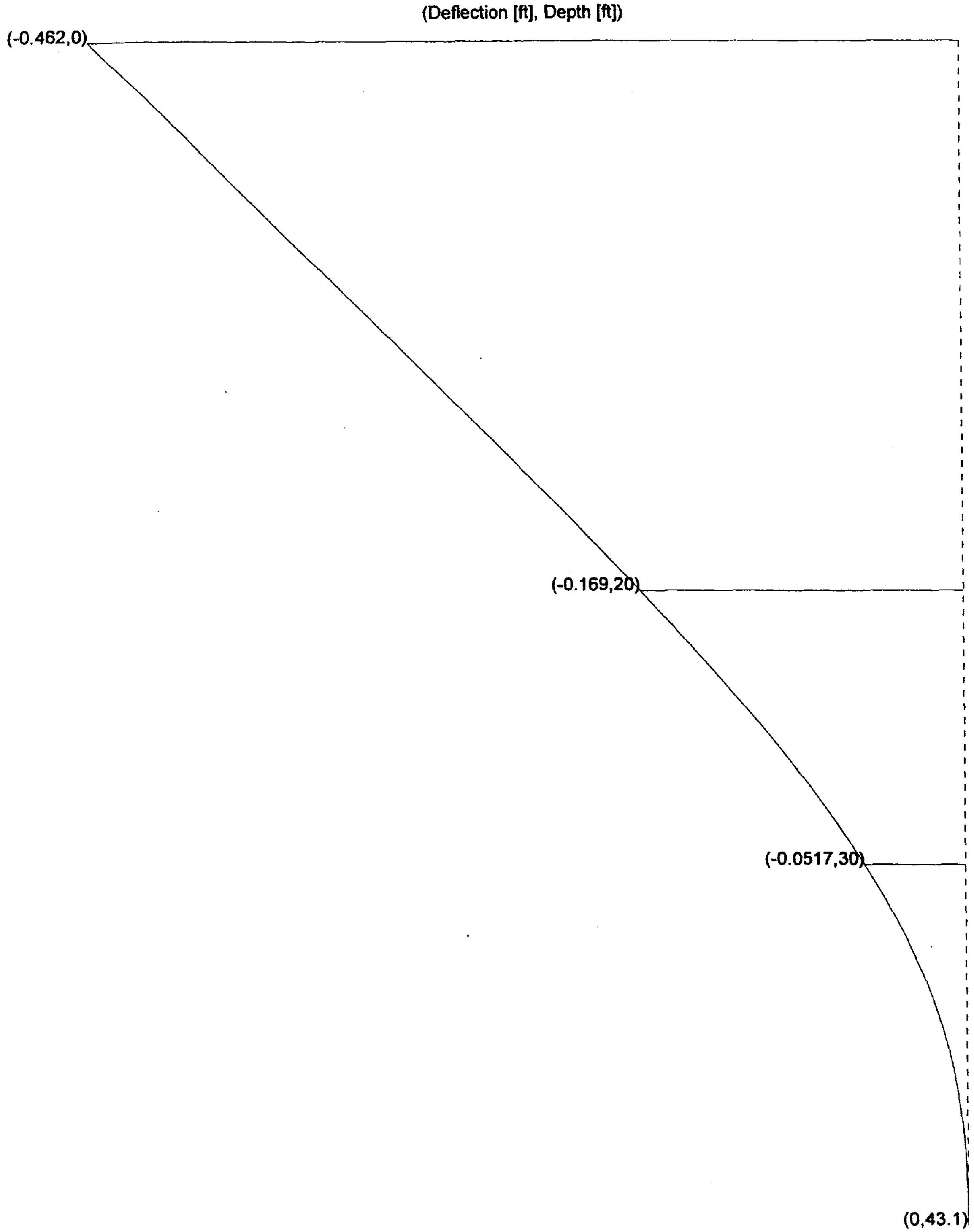
SCALE: 1" = 20'

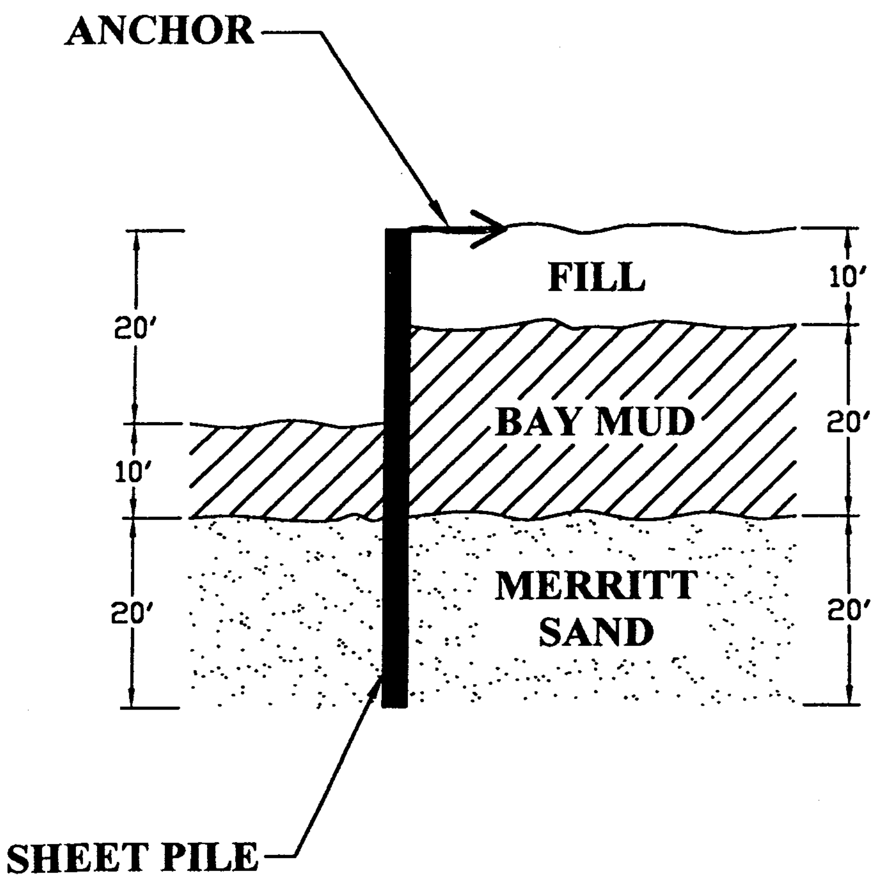
SECTION F-F'
CANTILIVERED SHEET PILE

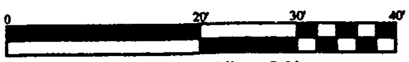
SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\FStatic.spc
Comment:	Section F-F' Post-EQ Soil Properties Cantilevered

DEFLECTION DIAGRAM







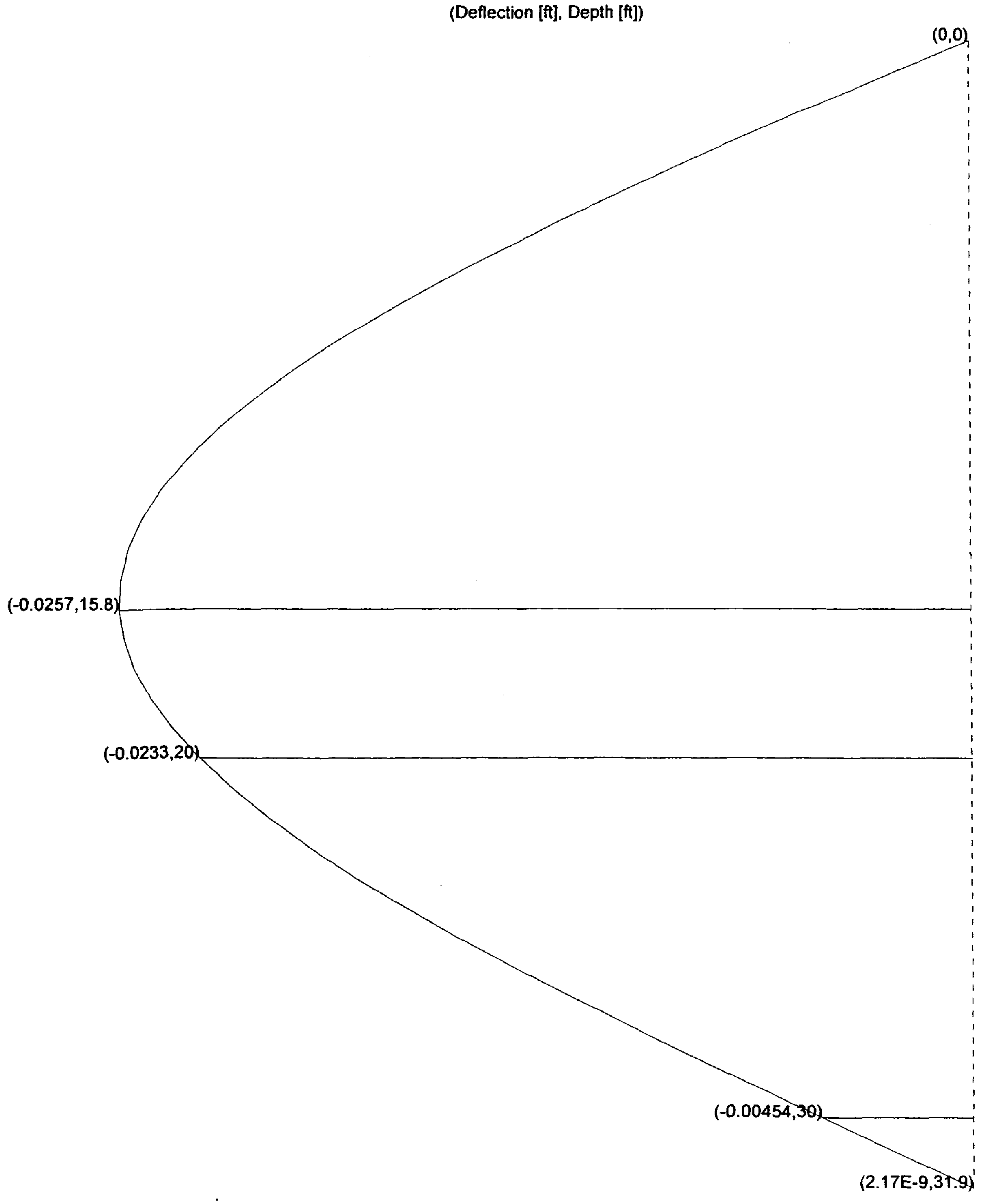
SCALE: 1" = 20'

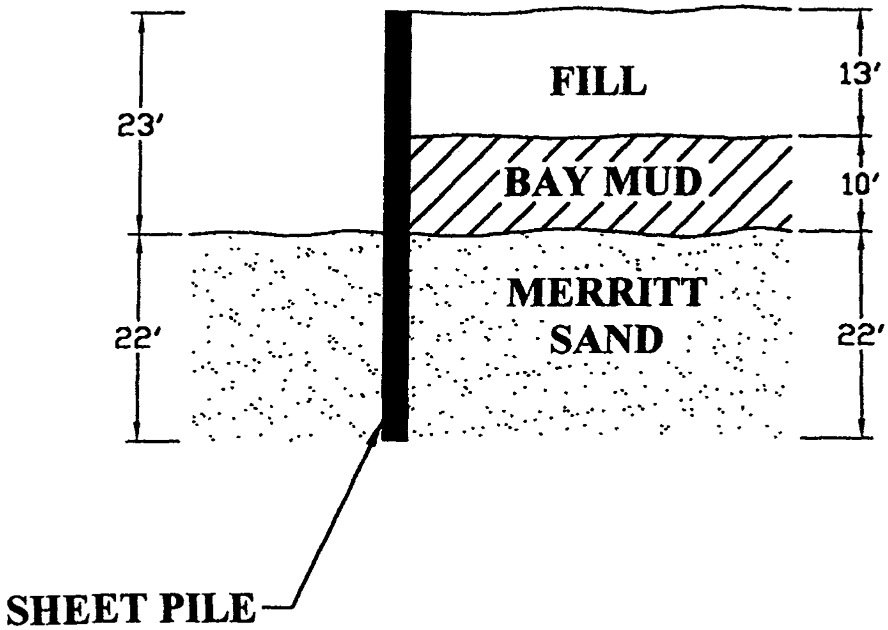
SECTION F-F'
ANCHORED SHEET PILE

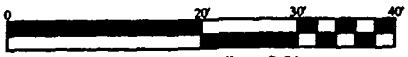
SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\FStatic_a.spc
Comment:	Section F-F' Post-EQ Soil Properties Anchored

DEFLECTION DIAGRAM







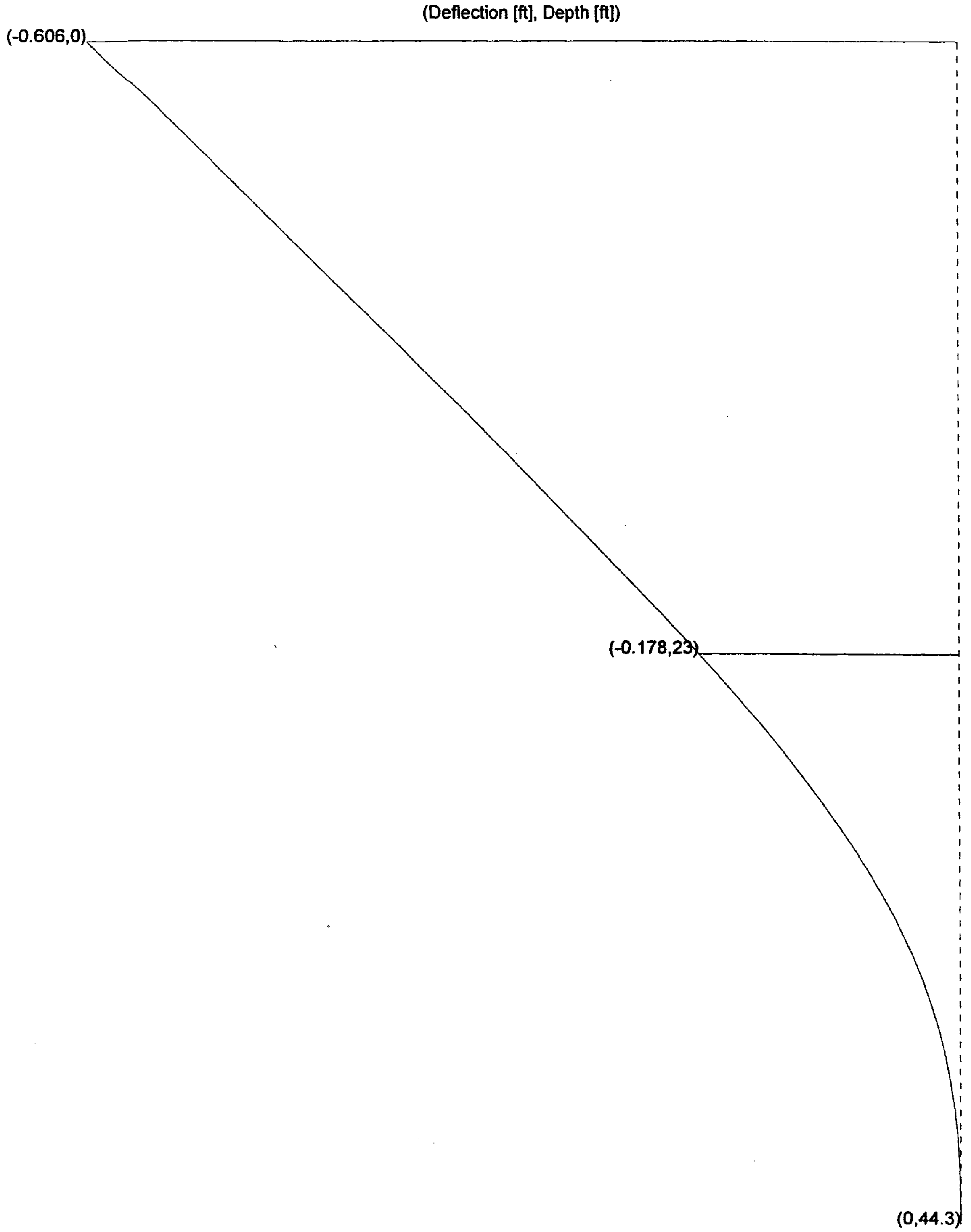
SCALE: 1" = 20'

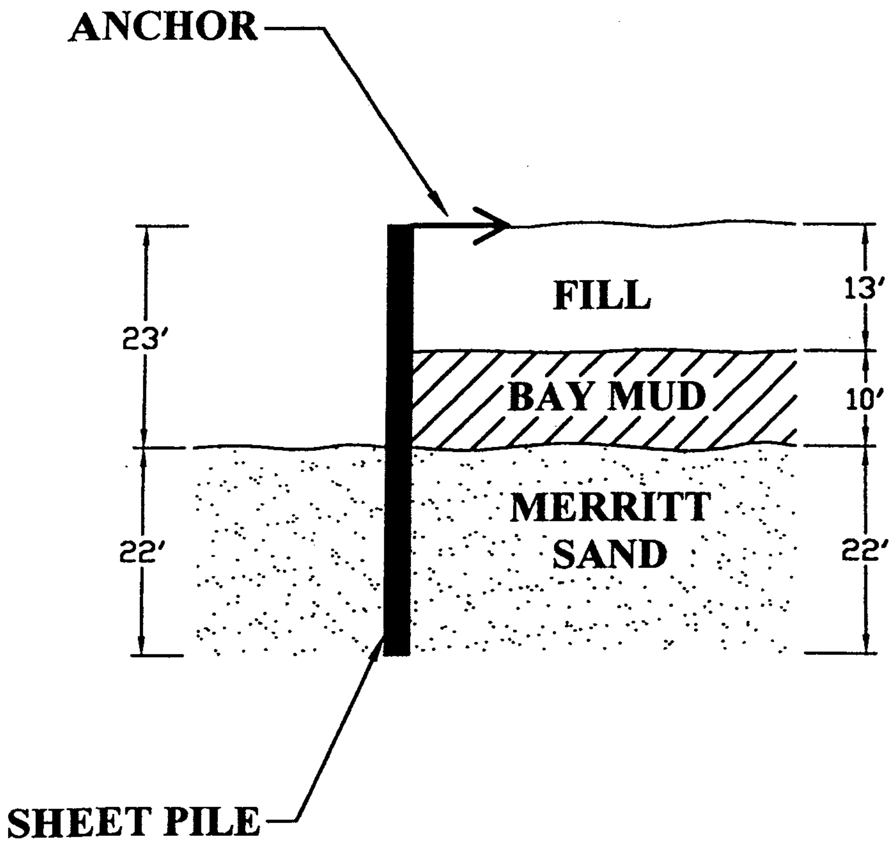
SECTION G-G'
CANTILIVERED SHEET PILE

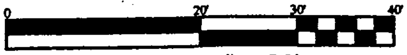
SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\GStatic.spc
Comment:	Section G-G' Post-EQ Soil Properties Cantilevered

DEFLECTION DIAGRAM







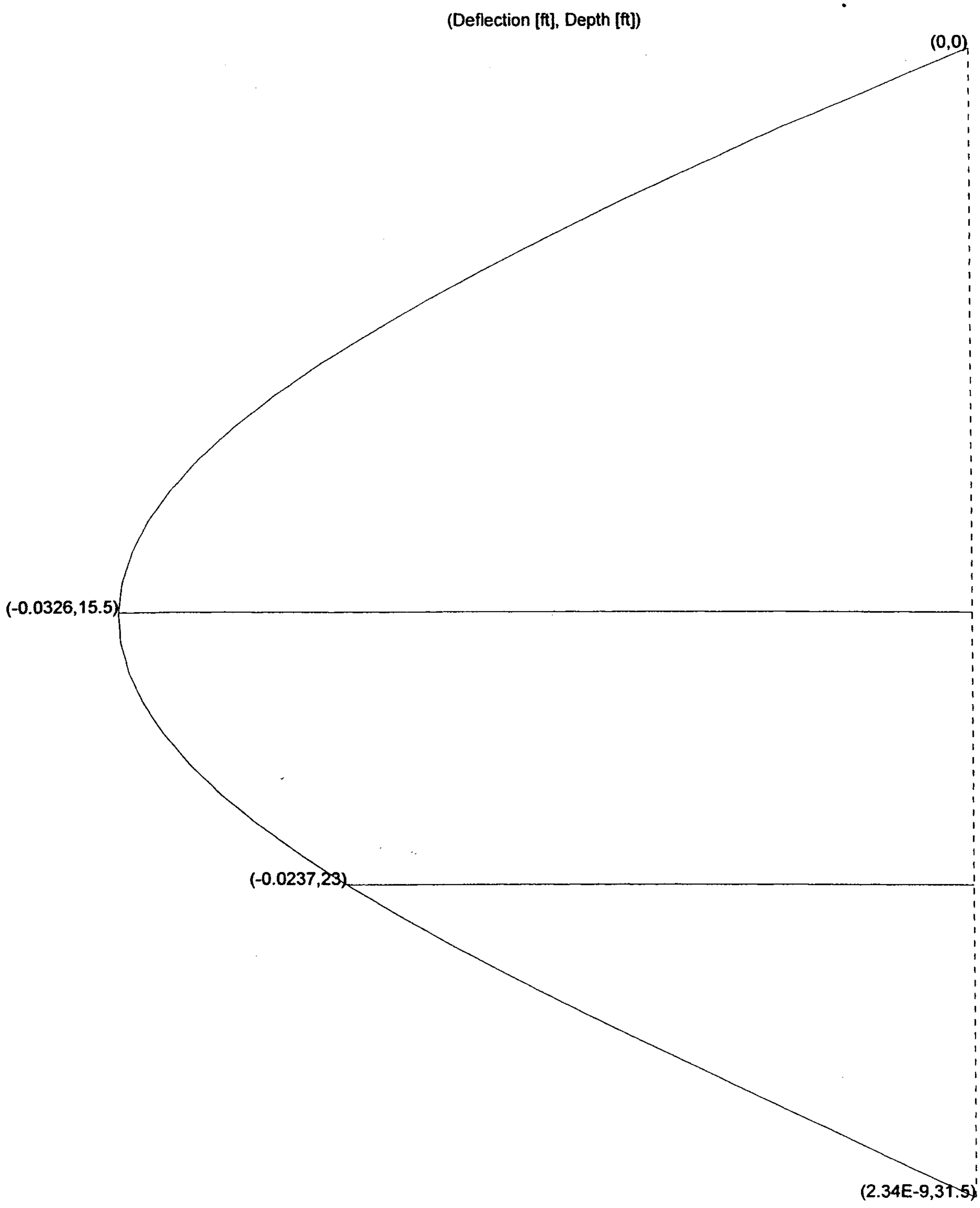
SCALE: 1" = 20'

SECTION G-G' **ANCHORED SHEET PILE**

SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

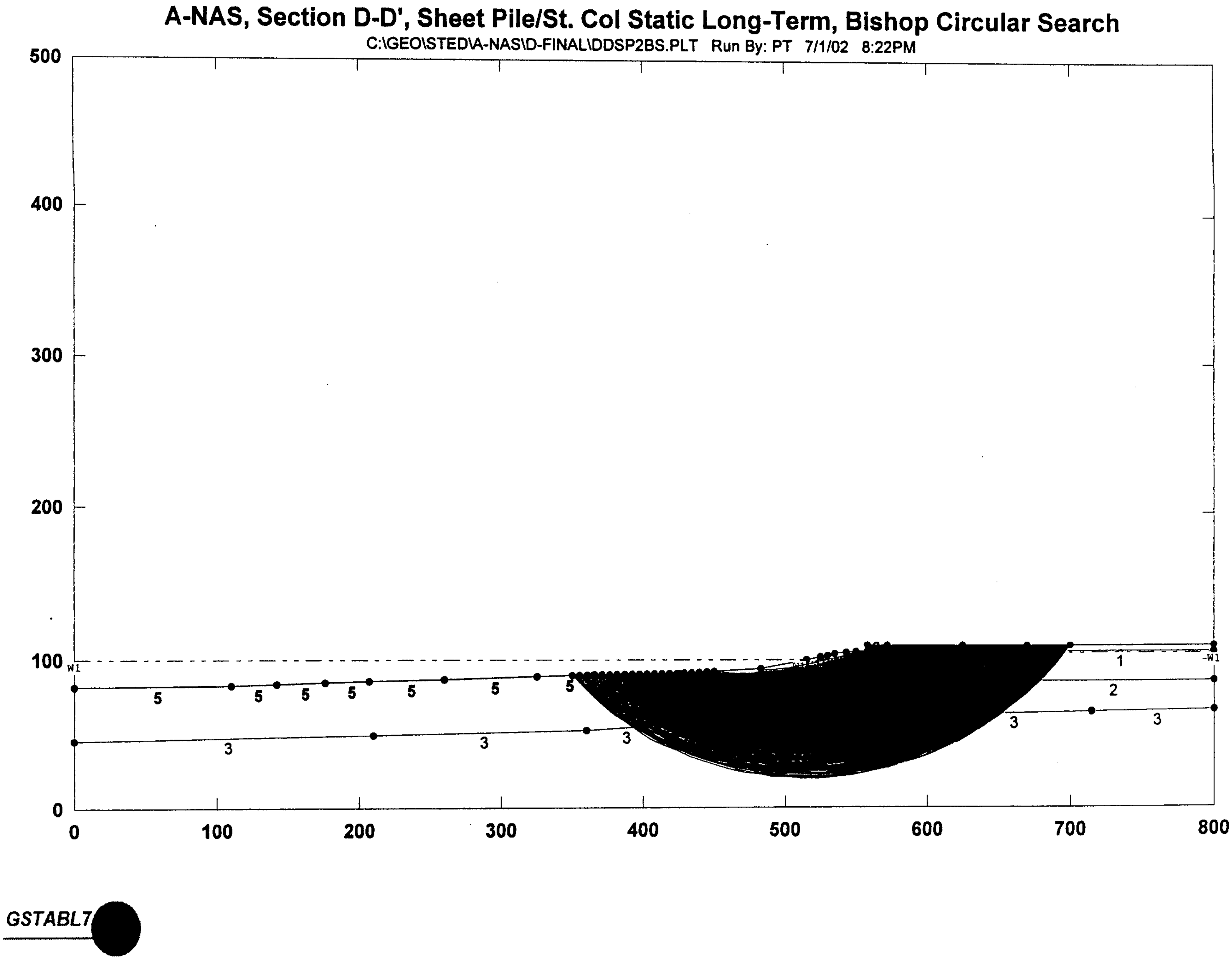
Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\GStatic_a.spc
Comment:	Section G-G' Post-EQ Soil Properties Anchored

DEFLECTION DIAGRAM



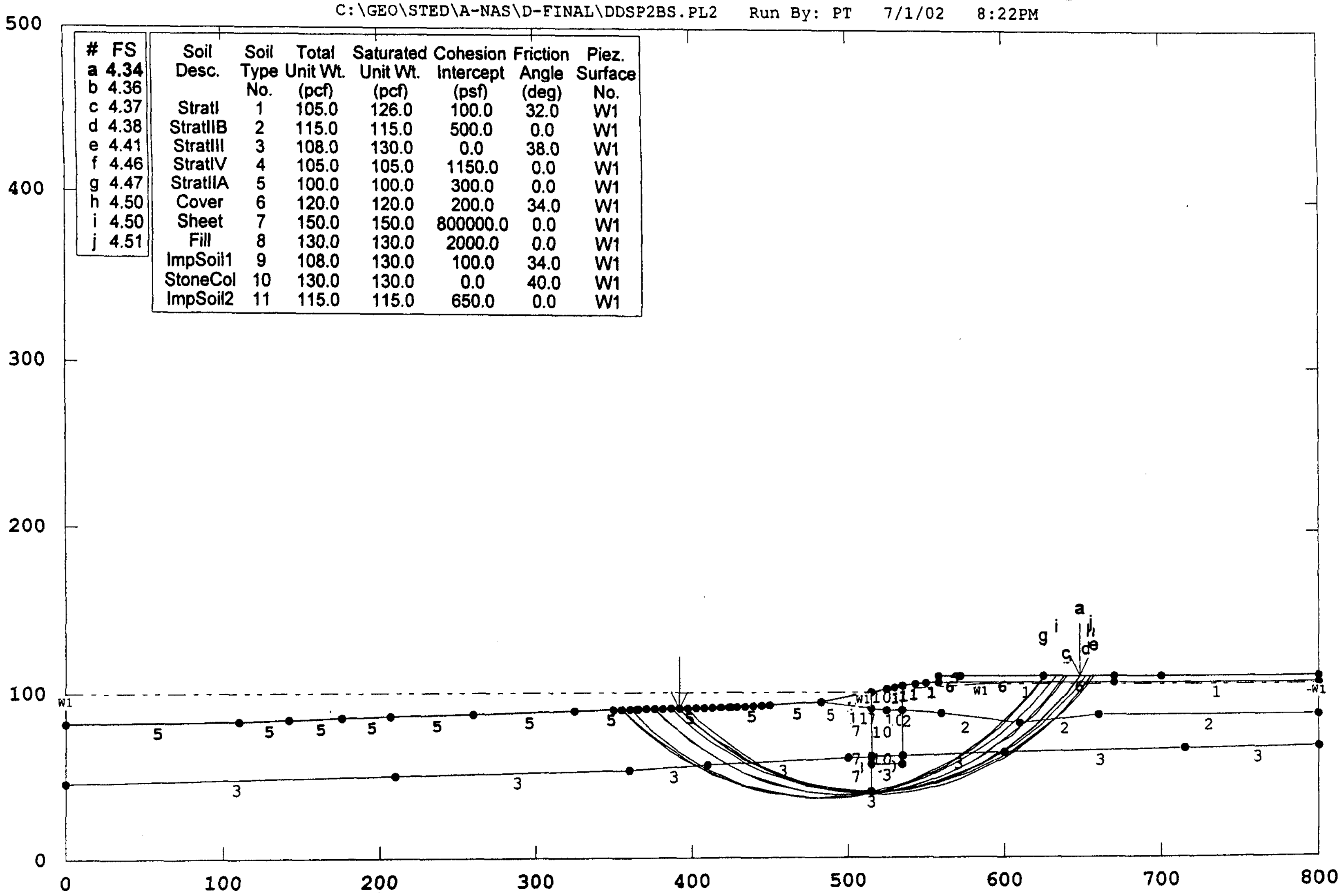
APPENDIX A4

ALTERNATIVE 4 – STONE COLUMNS WITH SURCHARGE AND SHEET PILES


A-NAS, Section D-D', Sheet Pile/St. Col Static Long-Term, Bishop Circular Search
C:\GEO\STED\A-NAS\D-FINAL\DDSP2BS.PLT Run By: PT 7/1/02 8:22PM
500
400
300
200
100
0
0
100
200
300
400
500
600
700
800
W1
5
3
1
2
GSTABL7

A-NAS, Section D-D', Sheet Pile/St. Col Static Long-Term, Bishop Circular Search

C:\GEO\STED\A-NAS\D-FINAL\DDSP2BS.PL2 Run By: PT 7/1/02 8:22PM



#	FS
a	4.34
b	4.36
c	4.37
d	4.38
e	4.41
f	4.46
g	4.47
h	4.50
i	4.50
j	4.51

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	100.0	32.0	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1150.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Sheet	7	150.0	150.0	800000.0	0.0	W1
Fill	8	130.0	130.0	2000.0	0.0	W1
ImpSoil1	9	108.0	130.0	100.0	34.0	W1
StoneCol	10	130.0	130.0	0.0	40.0	W1
ImpSoil2	11	115.0	115.0	650.0	0.0	W1

GSTABL7 v.2 FSmin=4.34

Safety Factors Are Calculated By The Modified Bishop Method

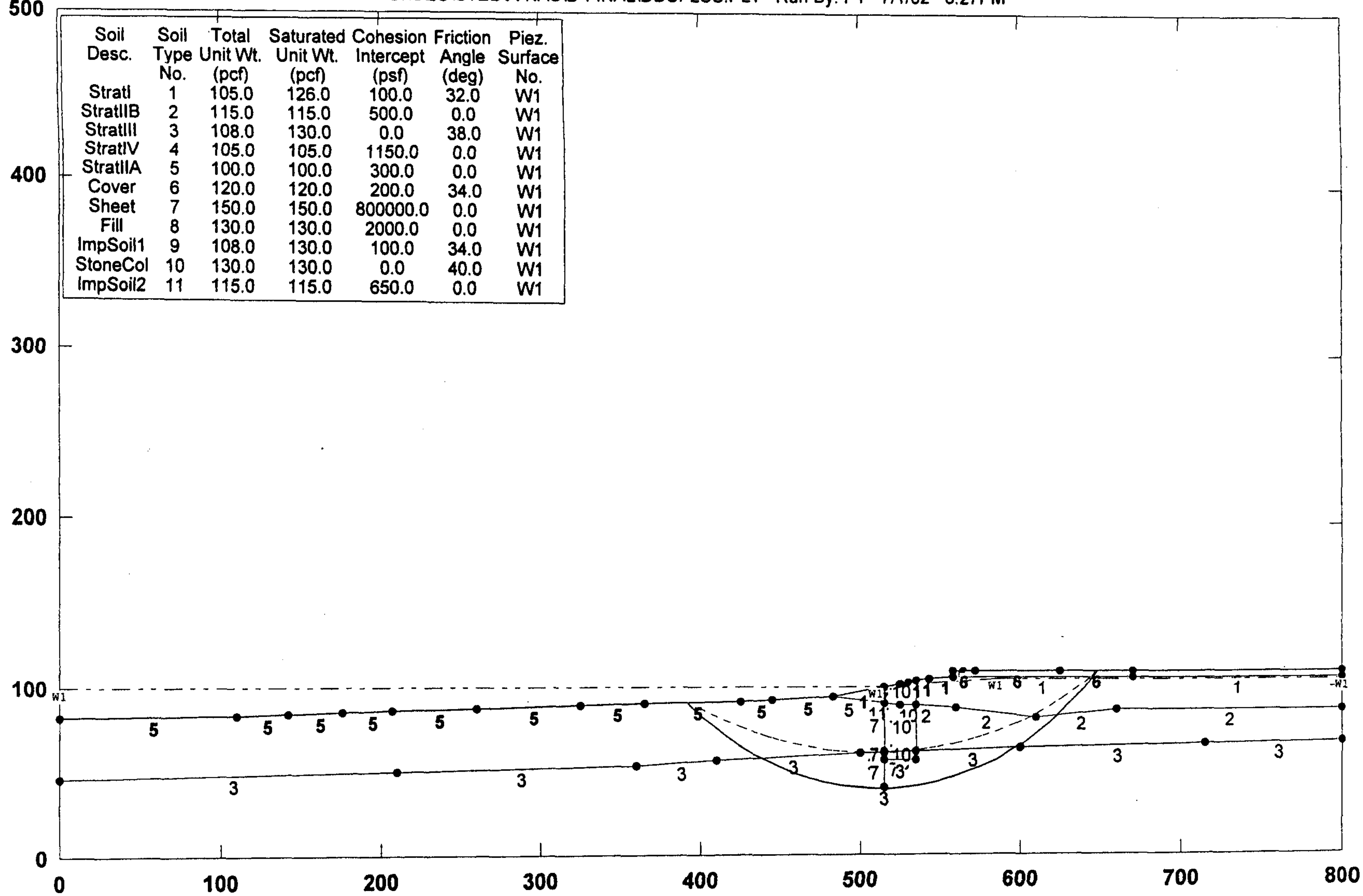
GSTABL7

```
PROFIL  c:\geo\sted\a-nas\d-final\ddsp2bs.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS, Section D-D', Sheet Pile/St. Col Static Long-Term, Bishop Circular Search
54 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 9
525. 102.1 530. 103.1 10
530. 103.1 535. 104.1 10
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
535. 104.1 535.21 88.66 10
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 525.21 89.33 11
525.21 89.33 535.21 88.66 10
535.21 88.66 560. 87. 2
535.21 88.66 535.31 61.1 10
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 535.31 61.1 10
535.31 61.1 600. 63. 3
535.31 61.1 535.4 56.1 10
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.33 55.5 7
515.33 55.5 535.4 56.1 3
```

```
515.33 55.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
ImpSoil1StoneColImpSoil2
11
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1150. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
115. 115. 650. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2
20 100
350. 450. 550. 700.
0. 10. 0. 0.
```

A-NAS, Section D-D', Sheet Pile/St. Col Spencer Static (Long-Term) Stab Analysis

C:\GEO\STED\A-NAS\D-FINAL\DDSP2SS.PLT Run By: PT 7/1/02 8:27PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	100.0	32.0	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1150.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Sheet	7	150.0	150.0	800000.0	0.0	W1
Fill	8	130.0	130.0	2000.0	0.0	W1
ImpSoil1	9	108.0	130.0	100.0	34.0	W1
StoneCol	10	130.0	130.0	0.0	40.0	W1
ImpSoil2	11	115.0	115.0	650.0	0.0	W1

GSTABL7 v.2 FSmin=4.39

Factor Of Safety Is Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

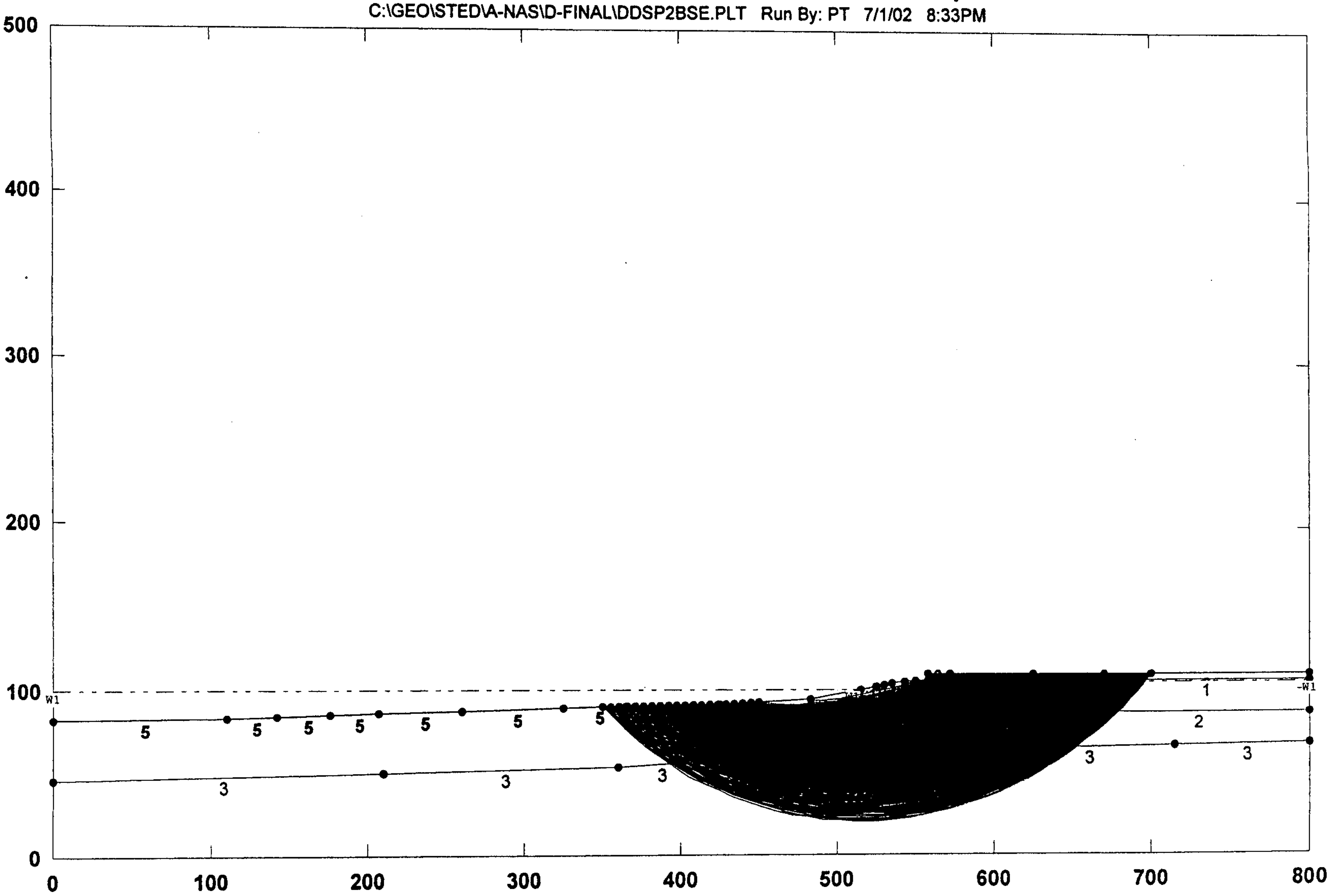
```
PROFIL  C:\geo\sted\a-nas\d-final\ddsp2ss.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS, Section D-D', Sheet Pile/St. Col Spencer Static (Long-Term) Stab Analysis
54 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 9
525. 102.1 530. 103.1 10
530. 103.1 535. 104.1 10
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
535. 104.1 535.21 88.66 10
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 525.21 89.33 11
525.21 89.33 535.21 88.66 10
535.21 88.66 560. 87. 2
535.21 88.66 535.31 61.1 10
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 535.31 61.1 10
535.31 61.1 600. 63. 3
535.31 61.1 535.4 56.1 10
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.33 55.5 7
515.33 55.5 535.4 56.1 3
```

```
515.33 55.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
ImpSoil1StoneColImpSoil2
11
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1150. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
115. 115. 650. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
GLEMS
10.
1       Water filled tension crack (0=no,1=yes)
0       Force Distribution (0=Single slice,1=Entire failure surf)
0       Select Method (0=Spencer,1=Morgenstern-Price)
2       ki function (Spencer=1 or 2, M-P=1, 2, 3, 4, or 5=user)
1.000   Lambda Coefficient (adjusts ki, 0.4 to 1.0)
0       Trial Lambda Adjustment option (0=no, 1=yes)
SURBIS
31
392.11 90.45
399.25 83.45
406.8 76.9
414.73 70.81
423.02 65.21
431.62 60.11
440.51 55.54
449.67 51.51
459.05 48.04
468.62 45.14
478.34 42.82
488.19 41.09
498.13 39.96
508.11 39.42
518.11 39.48
528.09 40.15
538.01 41.41
547.84 43.26
557.54 45.7
567.07 48.72
576.4 52.31
585.51 56.45
594.34 61.13
```

```
602.88 66.34
611.09 72.04
618.95 78.23
626.42 84.88
633.47 91.97
640.09 99.47
646.24 107.35
648.13 110.1
EXECUT
```

A-NAS, Section D-D', Sheet Pile/St. Col PostEQ Static Bishop Circular Search

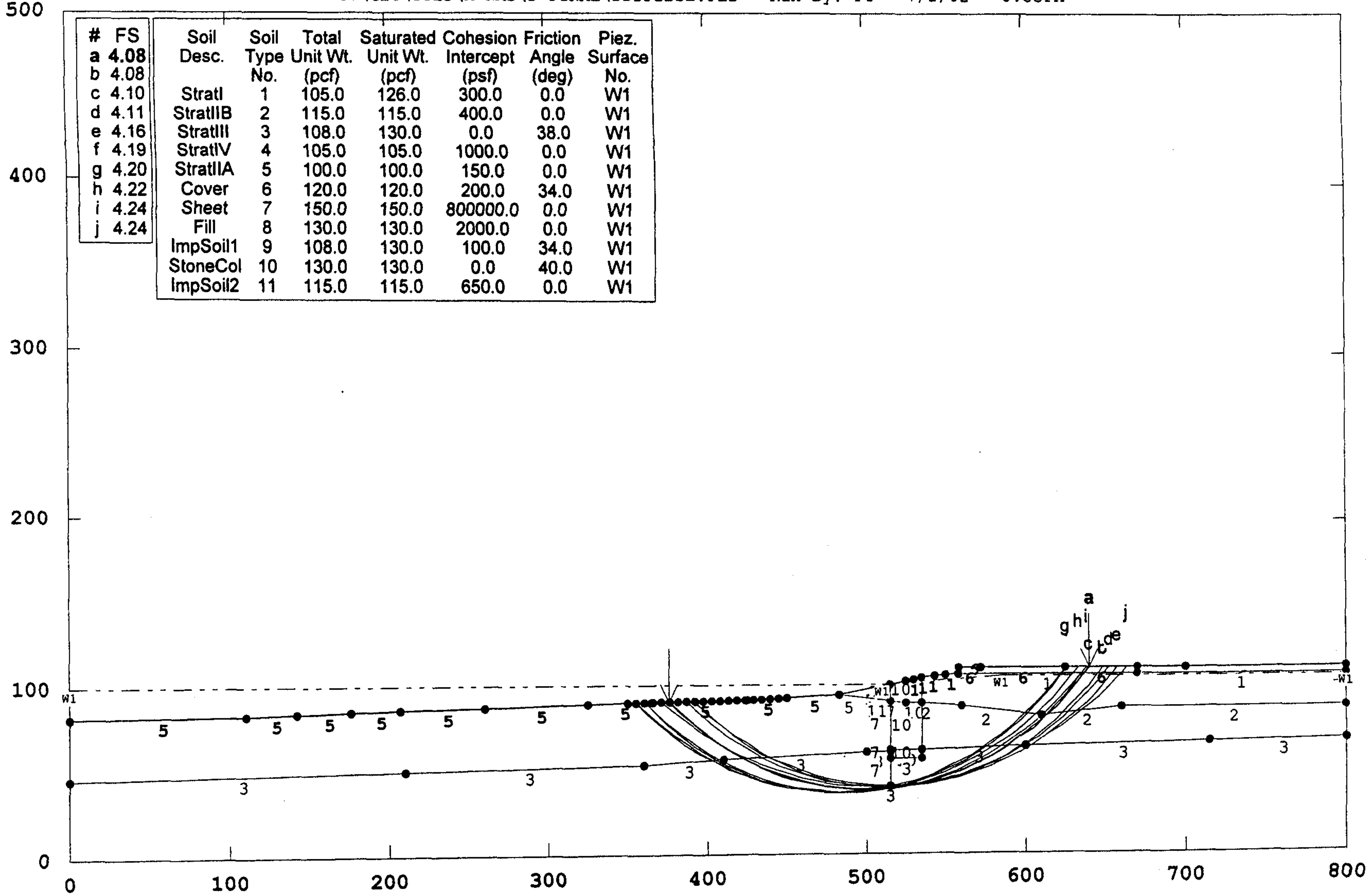
C:\GEO\STED\A-NAS\D-FINAL\DDSP2BSE.PLT Run By: PT 7/1/02 8:33PM



GSTABL7

A-NAS, Section D-D', Sheet Pile/St. Col PostEQ Static Bishop Circular Search

C:\GEO\STED\A-NAS\D-FINAL\DDSP2BSE.PL2   Run By: PT   7/1/02   8:33PM



#	FS
a	4.08
b	4.08
c	4.10
d	4.11
e	4.16
f	4.19
g	4.20
h	4.22
i	4.24
j	4.24

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	300.0	0.0	W1
StratIIB	2	115.0	115.0	400.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1000.0	0.0	W1
StratIIA	5	100.0	100.0	150.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Sheet	7	150.0	150.0	800000.0	0.0	W1
Fill	8	130.0	130.0	2000.0	0.0	W1
ImpSoil1	9	108.0	130.0	100.0	34.0	W1
StoneCol	10	130.0	130.0	0.0	40.0	W1
ImpSoil2	11	115.0	115.0	650.0	0.0	W1

GSTABL7 v.2 FSmin=4.08

Safety Factors Are Calculated By The Modified Bishop Method

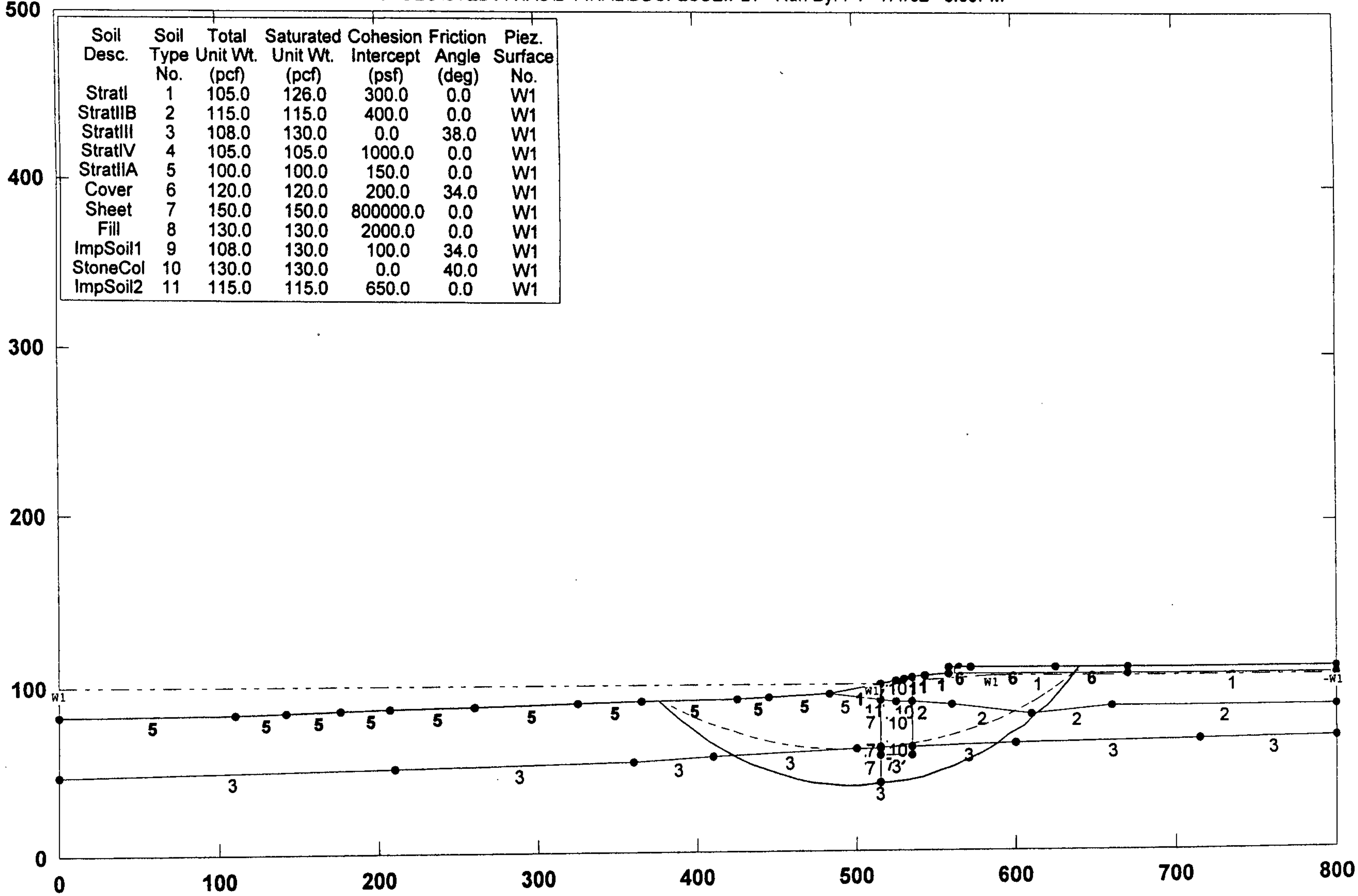
GSTABL7

```
PROFIL  c:\geo\sted\a-nas\d-final\ddsp2bse.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS, Section D-D', Sheet Pile/St. Col PostEQ Static Bishop Circular Search
54 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 9
525. 102.1 530. 103.1 10
530. 103.1 535. 104.1 10
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
535. 104.1 535.21 88.66 10
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 525.21 89.33 11
525.21 89.33 535.21 88.66 10
535.21 88.66 560. 87. 2
535.21 88.66 535.31 61.1 10
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 535.31 61.1 10
535.31 61.1 600. 63. 3
535.31 61.1 535.4 56.1 10
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.33 55.5 7
515.33 55.5 535.4 56.1 3
```

```
515.33 55.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
ImpSoil1StoneColImpSoil2
11
105. 126. 300. 0. 0. 0. 1
115. 115. 400. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 150. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
115. 115. 650. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2
20 100
350. 450. 550. 700.
0. 10. 0. 0.
```

A-NAS, Section D-D', Sheet Pile/St. Col PostEQ Spencer Static Stability Analysis

C:\GEO\STED\A-NAS\D-FINAL\DDSP2SSE.PLT Run By: PT 7/1/02 8:35PM



GSTABL7 v.2 FSmin=4.14

Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

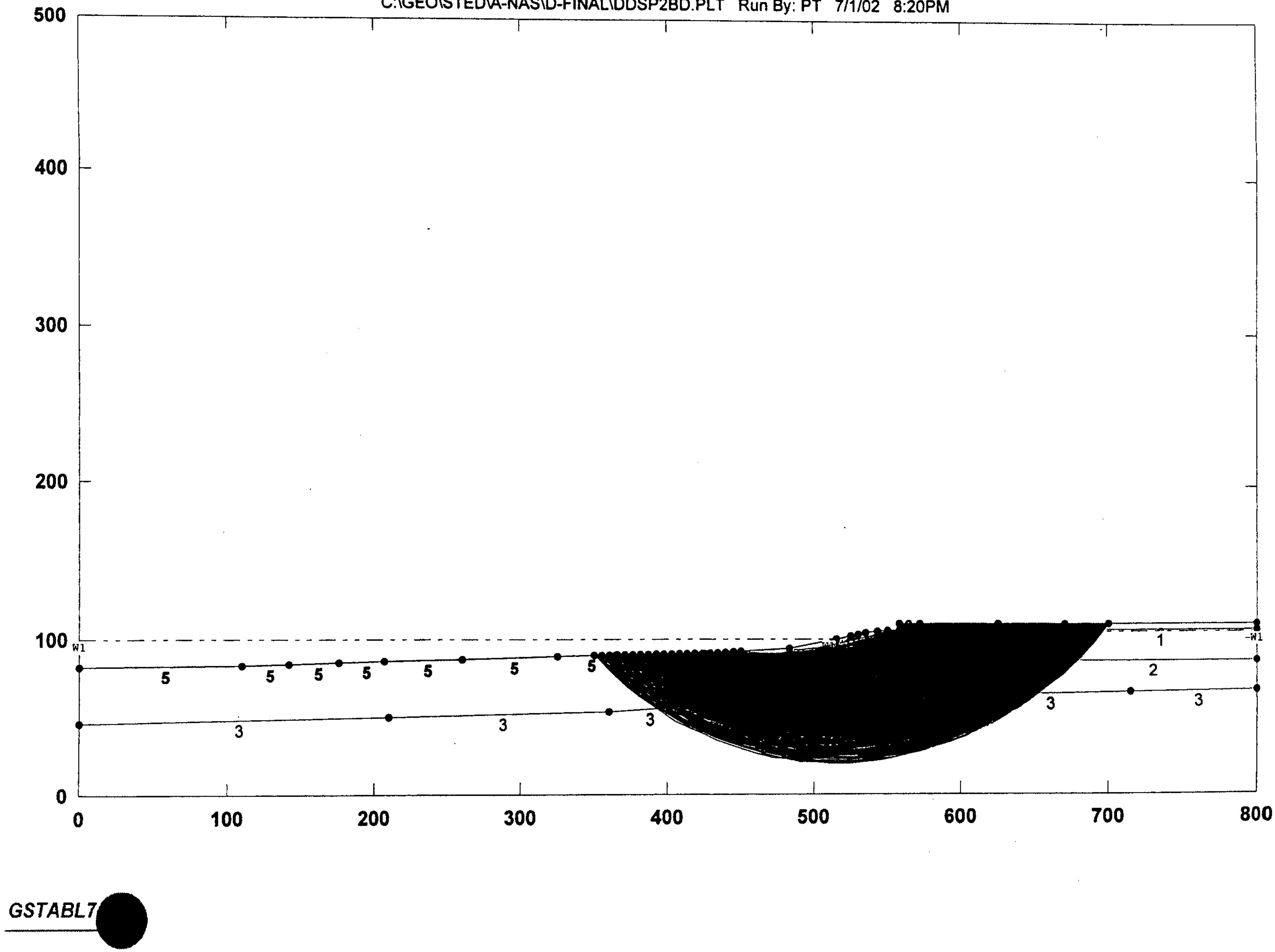
```
PROFIL  C:\geo\sted\a-nas\d-final\ddsp2sse.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS, Section D-D', Sheet Pile/St. Col PostEQ Spencer Static Stability Analysis
54 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 9
525. 102.1 530. 103.1 10
530. 103.1 535. 104.1 10
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
535. 104.1 535.21 88.66 10
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 525.21 89.33 11
525.21 89.33 535.21 88.66 10
535.21 88.66 560. 87. 2
535.21 88.66 535.31 61.1 10
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 535.31 61.1 10
535.31 61.1 600. 63. 3
535.31 61.1 535.4 56.1 10
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.33 55.5 7
515.33 55.5 535.4 56.1 3
```

```
515.33 55.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
ImpSoil1StoneColImpSoil2
11
105. 126. 300. 0. 0. 0. 1
115. 115. 400. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
100. 100. 150. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
115. 115. 650. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
GLEMS
10.
1       Water filled tension crack (0=no,1=yes)
0       Force Distribution (0=Single slice,1=Entire failure surf)
0       Select Method (0=Spencer,1=Morgenstern-Price)
2       ki function (Spencer=1 or 2, M-P=1, 2, 3, 4, or 5=user)
1.000   Lambda Coefficient (adjusts ki, 0.4 to 1.0)
0       Trial Lambda Adjustment option (0=no, 1=yes)
SURBIS
32
376.32 90.19
383.52 83.25
391.1 76.74
399.06 70.67
407.35 65.08
415.94 59.97
424.82 55.37
433.95 51.29
443.3 47.75
452.84 44.75
462.54 42.31
472.36 40.43
482.28 39.12
492.25 38.39
502.25 38.23
512.24 38.65
522.19 39.65
532.07 41.22
541.83 43.36
551.46 46.06
560.92 49.32
570.17 53.11
579.19 57.43
```

```
587.94 62.27
596.4 67.6
604.54 73.41
612.33 79.68
619.74 86.4
626.75 93.53
633.34 101.05
639.49 108.94
640.29 110.1
EXECUT
```

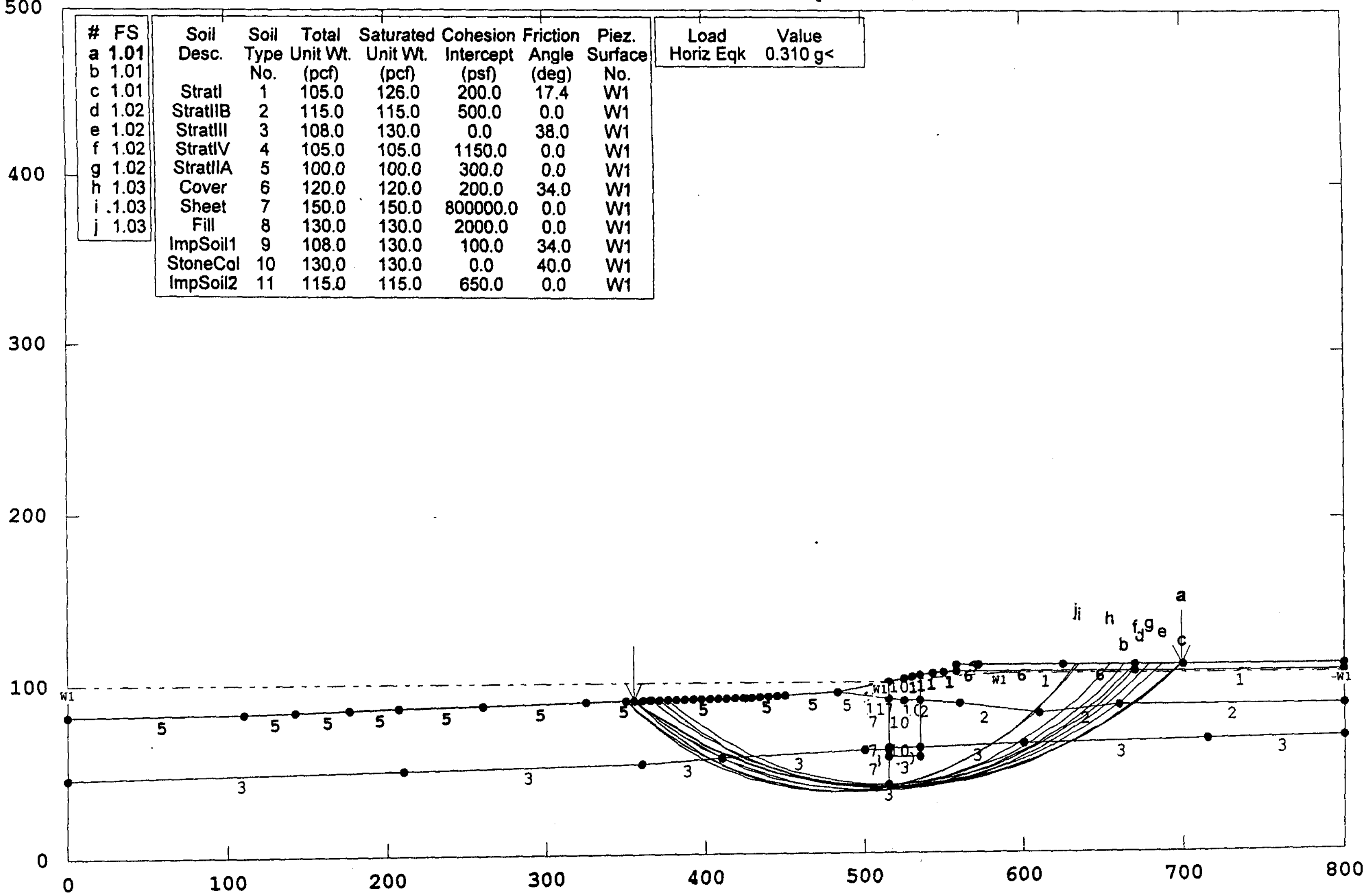
A-NAS, Section D-D', Sheet Pile/St. Col Dynamic Bishop Circular Search, 0.31g

C:\GEO\STED\A-NAS\D-FINAL\DDSP2BD.PLT Run By: PT 7/1/02 8:20PM



A-NAS, Section D-D', Sheet Pile/St. Col Dynamic Bishop Circular Search, 0.31g

C:\GEO\STED\A-NAS\D-FINAL\DDSP2BD.PL2 Run By: PT 7/1/02 8:20PM



#	FS
a	**1.01**
b	1.01
c	1.01
d	1.02
e	1.02
f	1.02
g	1.02
h	1.03
i	1.03
j	1.03

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	200.0	17.4	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1150.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Sheet	7	150.0	150.0	800000.0	0.0	W1
Fill	8	130.0	130.0	2000.0	0.0	W1
ImpSoil1	9	108.0	130.0	100.0	34.0	W1
StoneCol	10	130.0	130.0	0.0	40.0	W1
ImpSoil2	11	115.0	115.0	650.0	0.0	W1

Load	Value
Horiz Eqk	0.310 g<

GSTABL7 v.2 FSmin=1.01

Safety Factors Are Calculated By The Modified Bishop Method

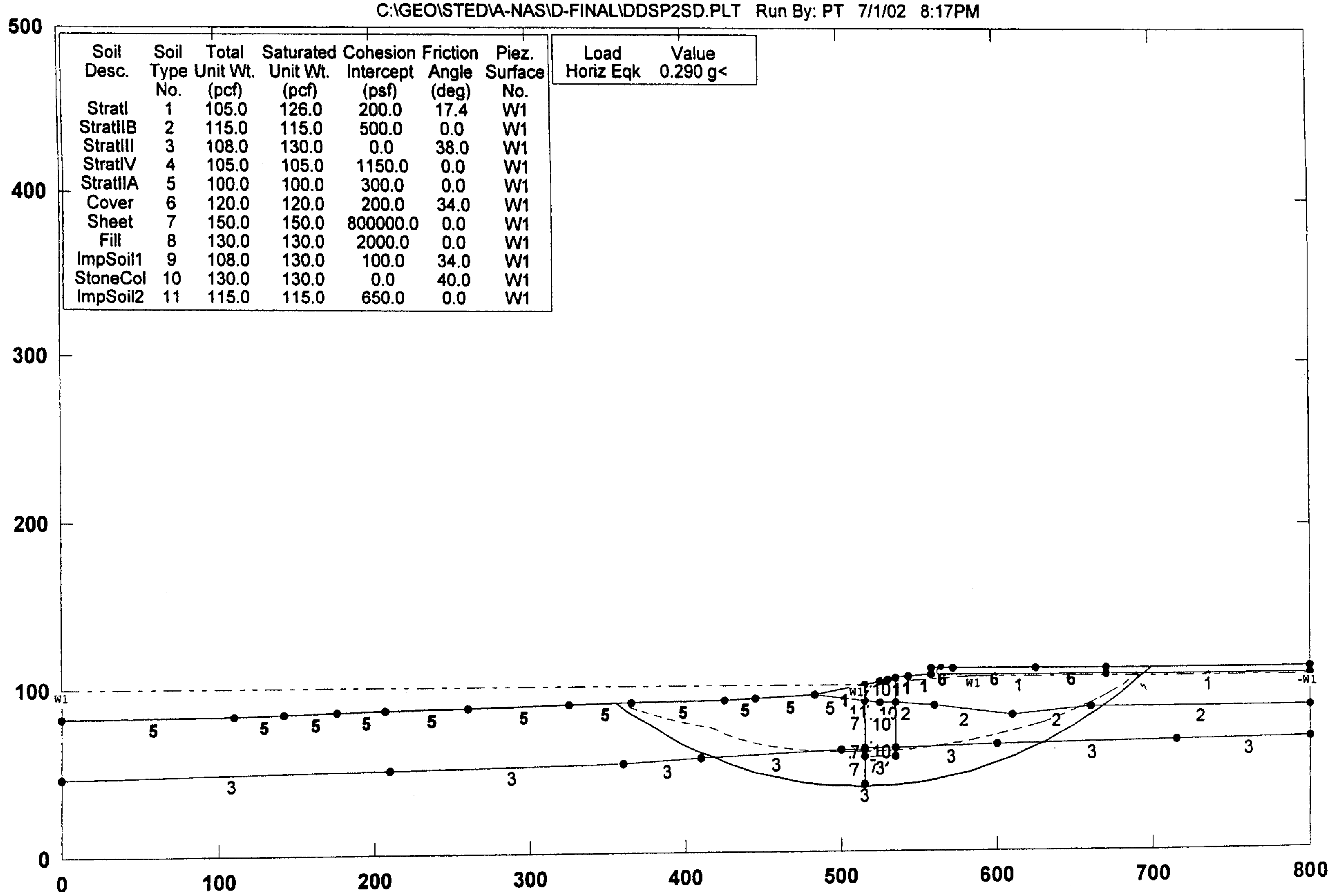
GSTABL7

```
PROFIL  c:\geo\sted\a-nas\d-final\ddsp2bd.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS, Section D-D', Sheet Pile/St. Col Dynamic Bishop Circular Search, 0.31g
54 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 9
525. 102.1 530. 103.1 10
530. 103.1 535. 104.1 10
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
535. 104.1 535.21 88.66 10
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 525.21 89.33 11
525.21 89.33 535.21 88.66 10
535.21 88.66 560. 87. 2
535.21 88.66 535.31 61.1 10
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 535.31 61.1 10
535.31 61.1 600. 63. 3
535.31 61.1 535.4 56.1 10
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.33 55.5 7
515.33 55.5 535.4 56.1 3
```

```
515.33 55.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
ImpSoil1StoneColImpSoil2
11
105. 126. 200. 17.4 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1150. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
115. 115. 650. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
EQUAKE
0.31 0. 0.
CIRCL2
20 100
350. 450. 550. 700.
0. 10. 0. 0.
```

A-NAS, Section D-D', Sheet Pile/St. Col Spencer Dynamic Slope Stability, 0.285g

C:\GEO\STED\A-NAS\D-FINAL\DDSP2SD.PLT Run By: PT 7/1/02 8:17PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	200.0	17.4	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1150.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
Sheet	7	150.0	150.0	800000.0	0.0	W1
Fill	8	130.0	130.0	2000.0	0.0	W1
ImpSoil1	9	108.0	130.0	100.0	34.0	W1
StoneCol	10	130.0	130.0	0.0	40.0	W1
ImpSoil2	11	115.0	115.0	650.0	0.0	W1

Load	Value
Horiz Eqk	0.290 g<

GSTABL7 v.2 FSmin=0.99

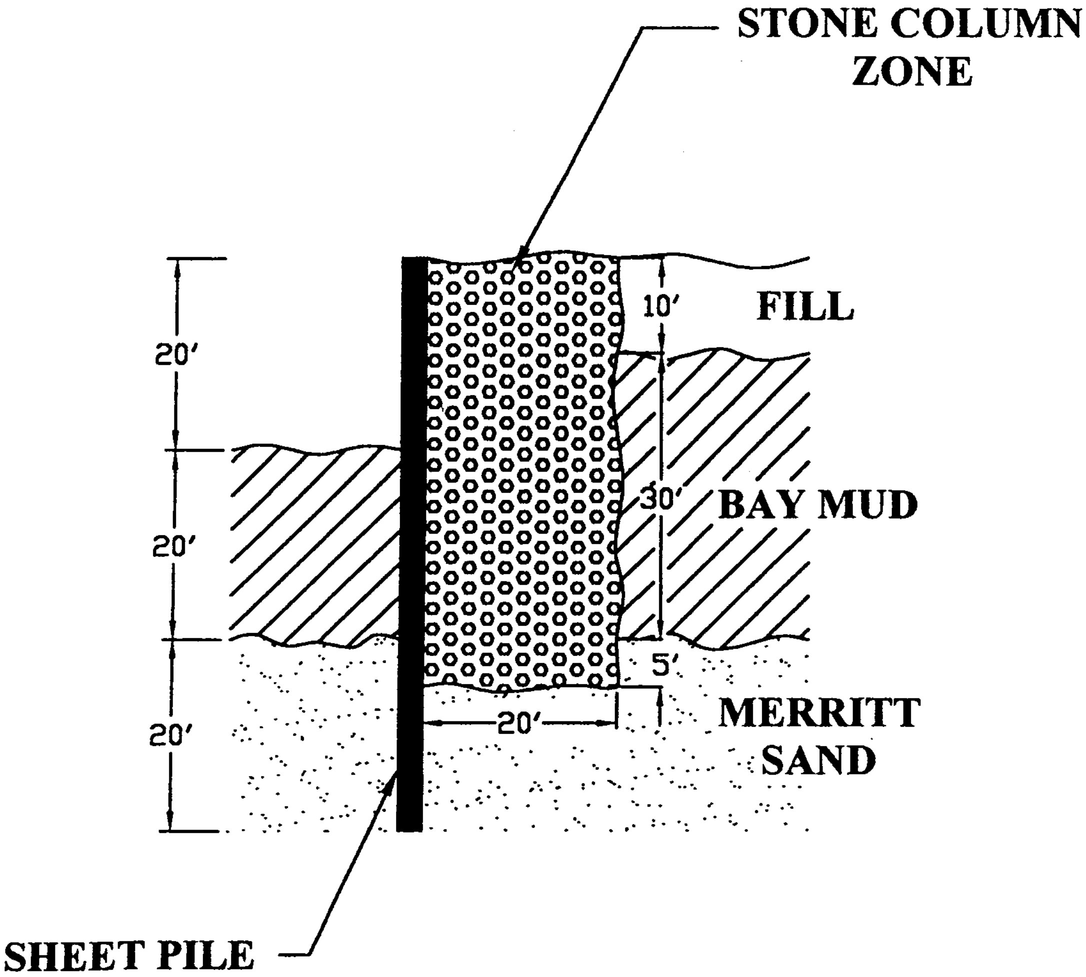
Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

```
PROFIL  C:\geo\sted\a-nas\d-final\ddsp2sd.in Version G7v.2 [GSTABL72.EXE] /O(0,
0)
e
A-NAS, Section D-D', Sheet Pile/St. Col Spencer Dynamic Slope Stability, 0.285g
54 22
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 515.11 100.2 7
515.11 100.2 525. 102.1 9
525. 102.1 530. 103.1 10
530. 103.1 535. 104.1 10
535. 104.1 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 7
515.11 100.2 515.21 90. 7
535. 104.1 535.21 88.66 10
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 515.21 90. 7
515.21 90. 525.21 89.33 11
525.21 89.33 535.21 88.66 10
535.21 88.66 560. 87. 2
535.21 88.66 535.31 61.1 10
515.21 90. 515.31 60.5 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 515.31 60.5 7
515.31 60.5 535.31 61.1 10
535.31 61.1 600. 63. 3
535.31 61.1 535.4 56.1 10
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 40. 514.9 60.45 7
515.31 60.5 515.33 55.5 7
515.33 55.5 535.4 56.1 3
```

```
515.33 55.5 515.41 40. 7
514.8 40. 515.41 40. 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   Sheet   Fill
ImpSoil1StoneColImpSoil2
11
105. 126. 200. 17.4 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1150. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
150. 150. 800000. 0. 0. 0. 1
130. 130. 2000. 0. 0. 0. 1
108. 130. 100. 34. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
115. 115. 650. 0. 0. 0. 1
WATER
1 0.
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
EQUAKE
0.29 0. 0.
GLEMS
10.
1       Water filled tension crack (0=no,1=yes)
0       Force Distribution (0=Single slice,1=Entire failure surf)
0       Select Method (0=Spencer,1=Morgenstern-Price)
2       ki function (Spencer=1 or 2, M-P=1, 2, 3, 4, or 5=user)
1.000   Lambda Coefficient (adjusts ki, 0.4 to 1.0)
0       Trial Lambda Adjustment option (0=no, 1=yes)
SURBIS
39
355.26 89.76
363.48 84.06
371.9 78.67
380.52 73.59
389.31 68.83
398.27 64.39
407.39 60.29
416.66 56.53
426.06 53.1
435.57 50.03
445.19 47.31
454.91 44.94
464.7 42.93
474.57 41.28
484.48 39.99
494.44 39.07
504.43 38.52
514.42 38.33
524.42 38.51
534.41 39.05
544.37 39.96
```

```
554.28 41.24
564.15 42.88
573.95 44.88
583.66 47.24
593.29 49.95
602.81 53.02
612.21 56.43
621.47 60.19
630.6 64.28
639.57 68.71
648.37 73.46
656.98 78.53
665.41 83.91
673.63 89.6
681.64 95.59
689.43 101.87
696.97 108.43
698.64 109.99
EXECUT
```



SCALE: 1" = 20'

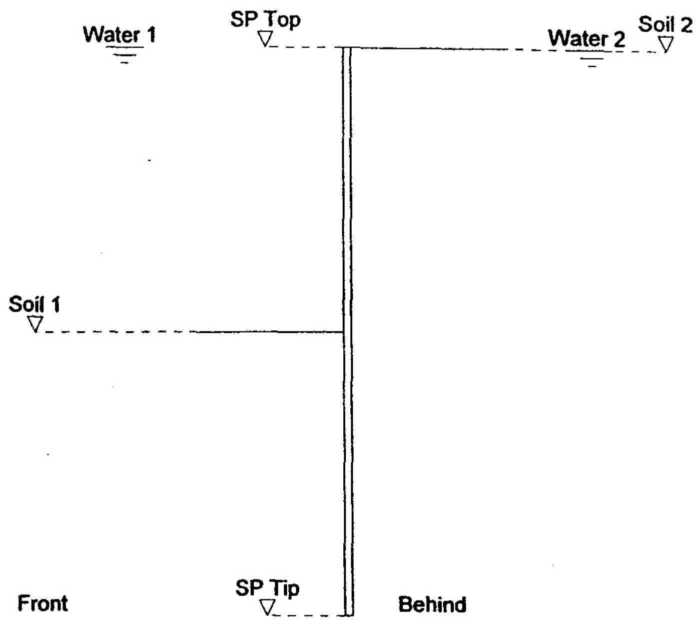
SECTION D-D' - STONE COLUMN AND SHEET PILE
-CANTILIVER-

SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/15/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\DStatic_sc.spc
Comment:	Section D-D' Post EQ Soil Properties 20 ft Stone Column Zone Cantilevered

GEODATA

Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	39.659
Soil Level in Front [ft]	20.000
Soil Level behind [ft]	0.000
Anchorlevel [ft]	0.000
Water Level in Front [ft]	0.000
Water Level behind [ft]	0.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	0.000
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Cantilever



LAYERS IN FRONT

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	40.000	0.115	0.052	1.000	0.000	0.000	0.400
Layer 2	120.000	0.130	0.067	4.208	38.000	0.000	0.000

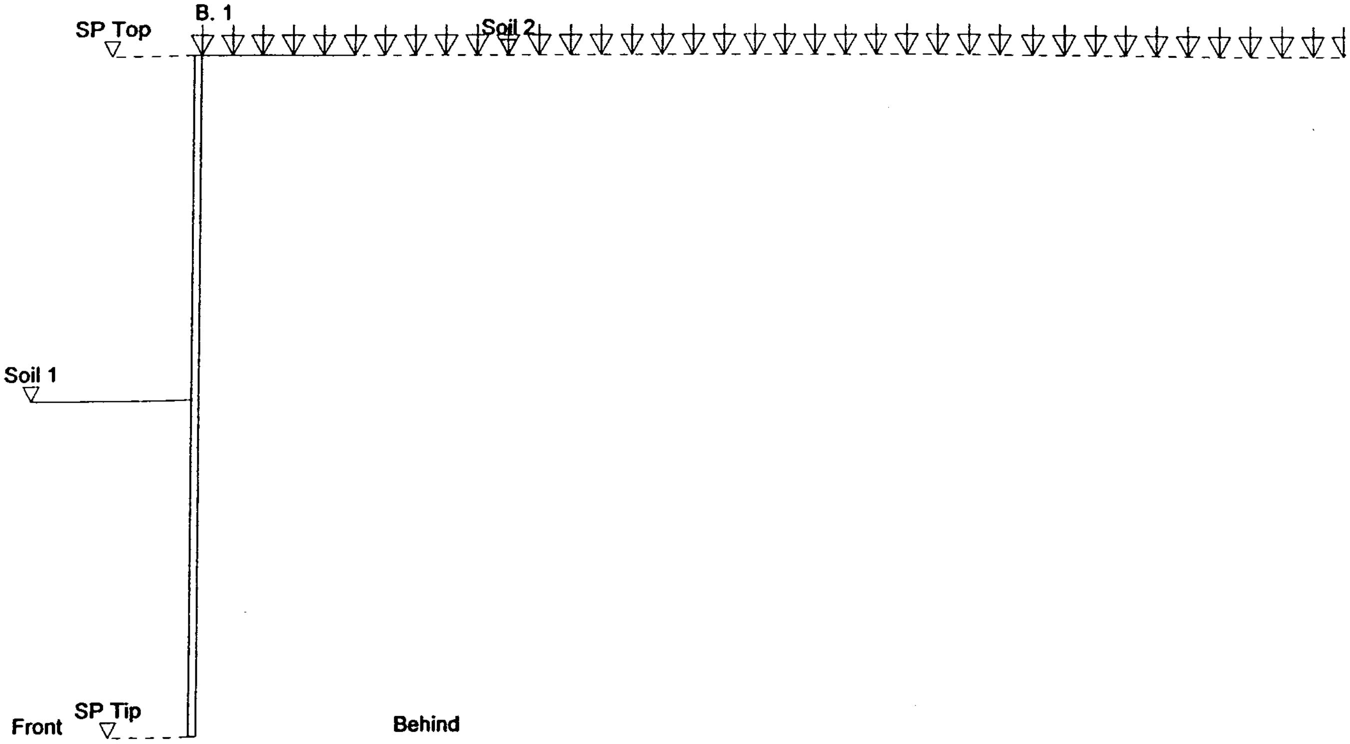
LAYERS BEHIND

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kah	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	10.000	0.128	0.066	1.000	0.000	0.000	0.300
Layer 2	17.800	0.126	0.066	1.000	0.000	0.000	0.400
Layer 3	40.000	0.130	0.068	0.217	40.000	0.000	0.400
Layer 4	45.000	0.130	0.680	0.217	40.000	0.000	0.000
Layer 5	120.000	0.130	0.680	0.238	38.000	0.000	0.000

49

BOUSSINESQ

	Distance Wall [ft]	Width Surcharge [ft]	Depth Surcharge [ft]	Surcharge [kip/ft2]
Bousq. 1	0.000	300.000	0.000	0.480



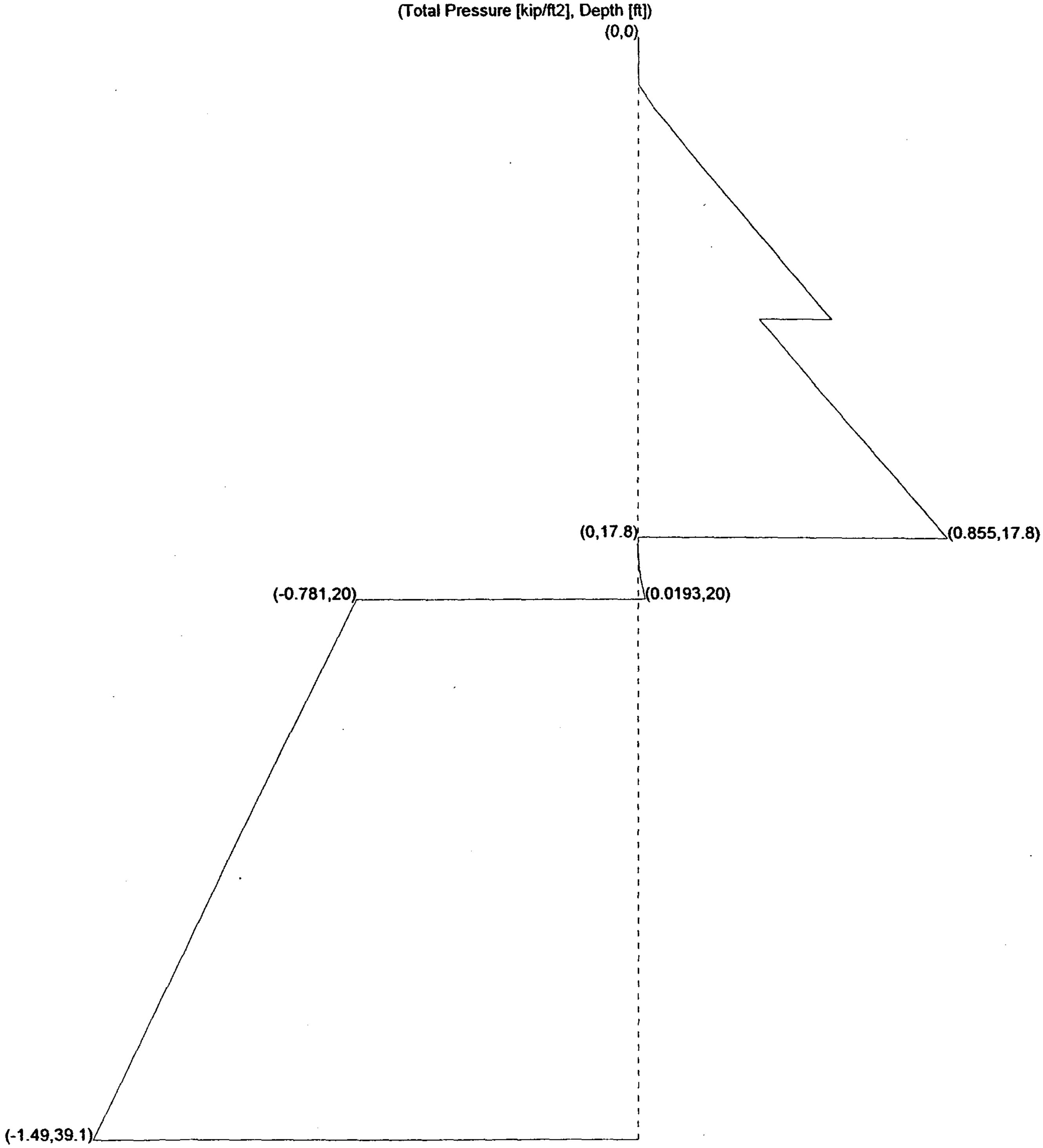
PILE SECTION

Name	AZ48
Inertia [in4/ft]	847.024
Modulus [in3/ft]	89.280
Area [in2/ft]	14.481
Mass [lbs/ft2]	49.279
Steelgrade [lb/in2]	60000.003
Requested Safety	2.000

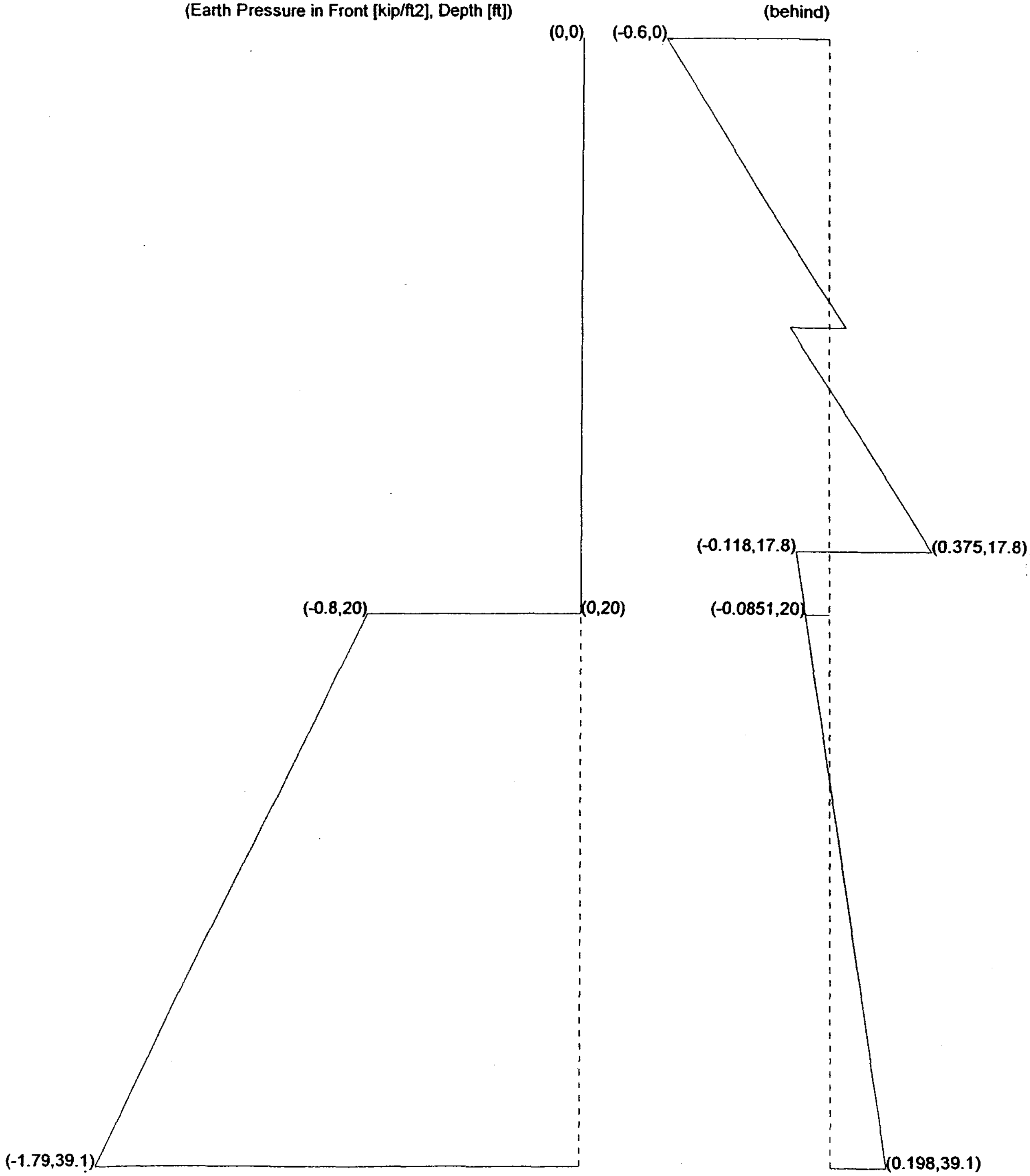
PILE CHECK

		Depth [ft]
Chosen Sheet Pile Section	AZ48	
Moment of Inertia [in4/ft]	847.024	
Section Modulus [in3/ft]	89.280	
Area [in2/ft]	14.481	
Mass [lbs/ft2]	49.279	
Steel Grade [lb/in2]	60000.003	
Minimal Moment [kipft/ft]	-0.902	39.154
Maximal Moment [kipft/ft]	81.067	27.487
Normal Forces at Min. Moment [kip/ft]	0.000	39.154
Normal Forces at Max. Moment [kip/ft]	0.000	27.487
Deflection at Min. Moment [ft]	0.000	39.154
Deflection at Max. Moment [ft]	-0.018	27.487
Min. Stress at Min. Moment [lb/in2]	-121.249	39.154
Max. Stress at Min. Moment [lb/in2]	121.249	39.154
Min. Stress at Max. Moment [lb/in2]	10895.654	27.487
Max. Stress at Max. Moment [lb/in2]	10895.654	27.487
Safety > Req. Safety = 2.000	5.507	
Pile Top [ft]		0.000
Pile Tip [ft]		39.659
Vertical Equilibrium [kip/ft]	0.000	
Anchor Force (horiz.) [kip/ft]	0.000	0.000

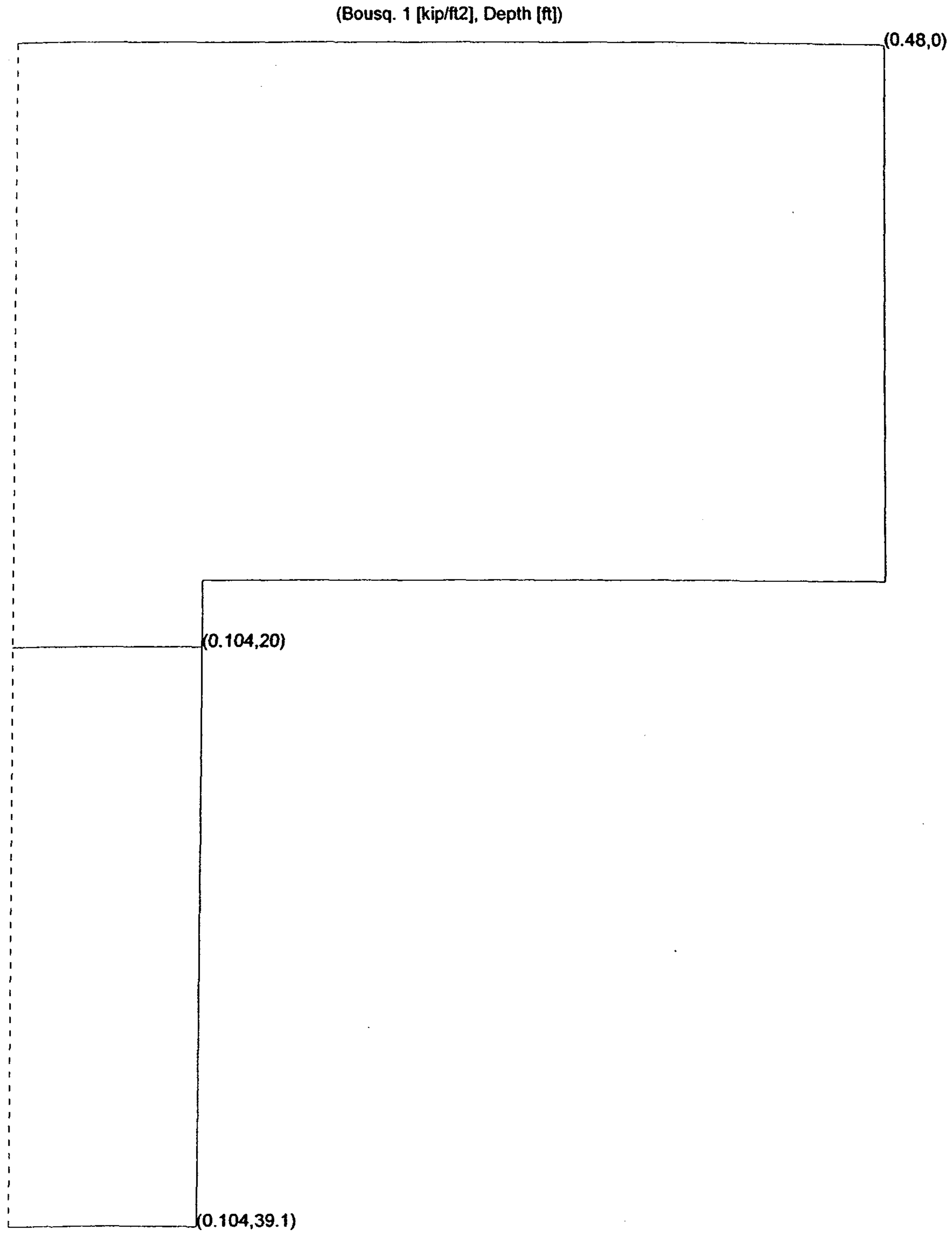
TOTAL PRESSURE DIAGRAM



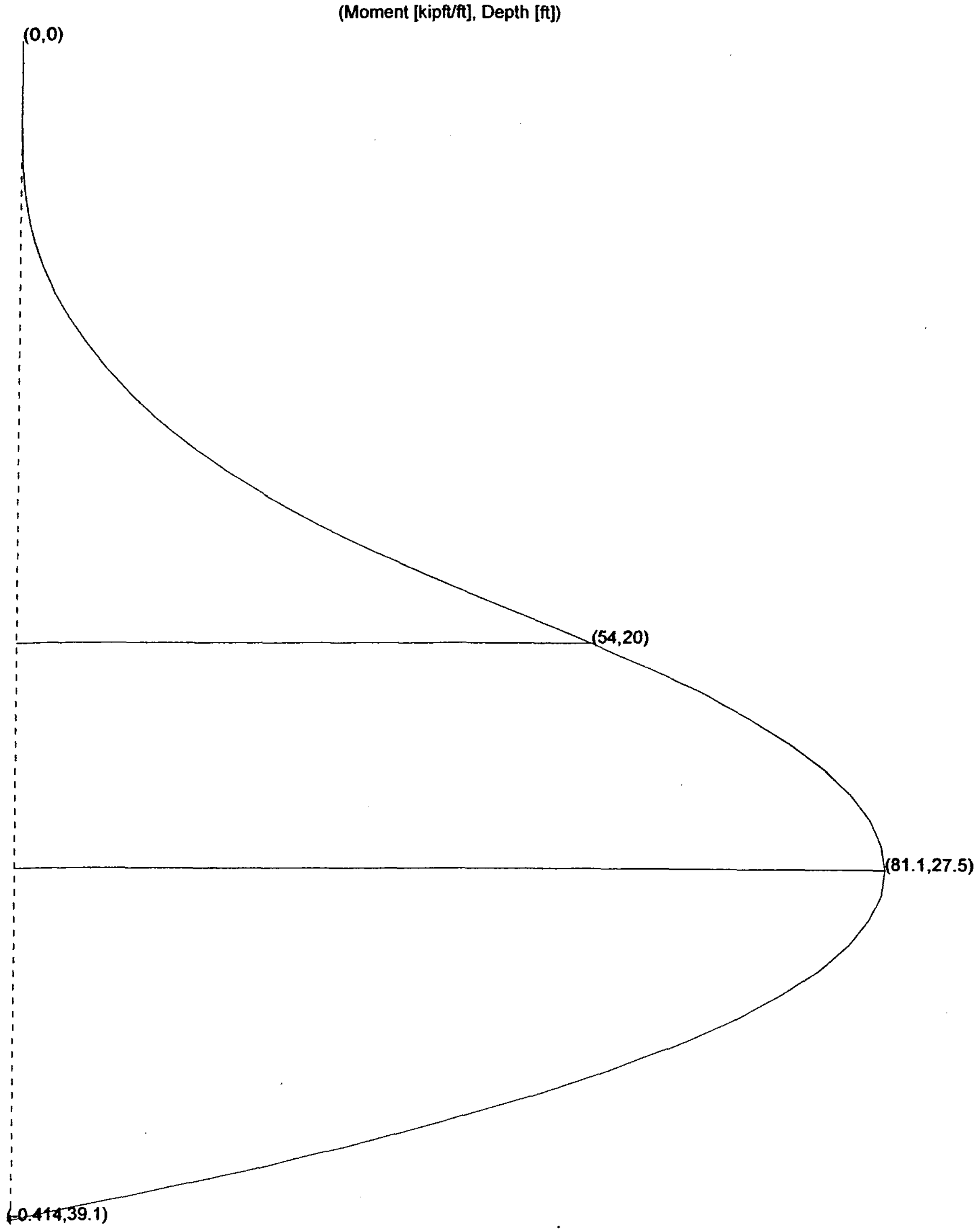
EARTH PRESSURE DIAGRAM



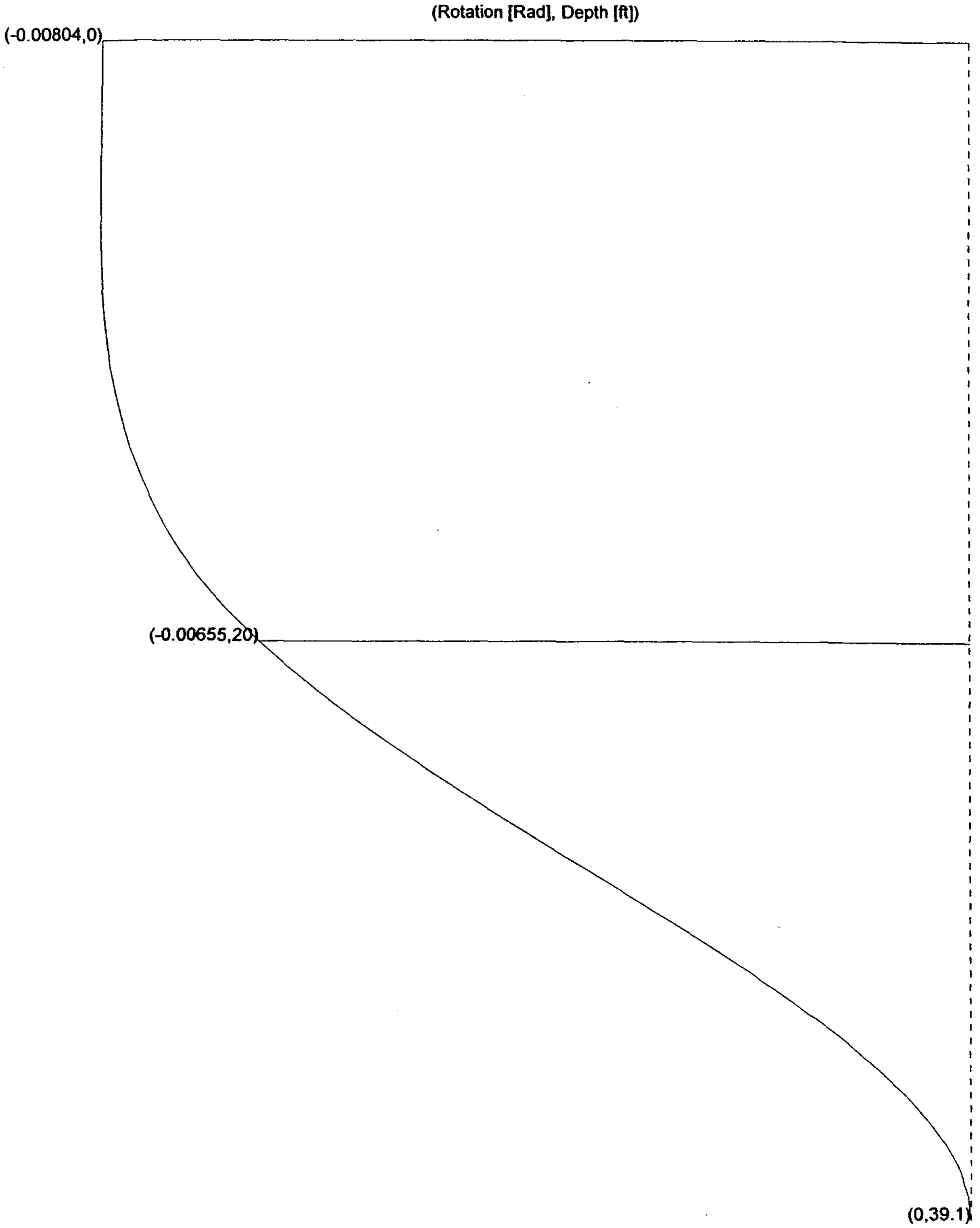
BOUSSINESQ DIAGRAM



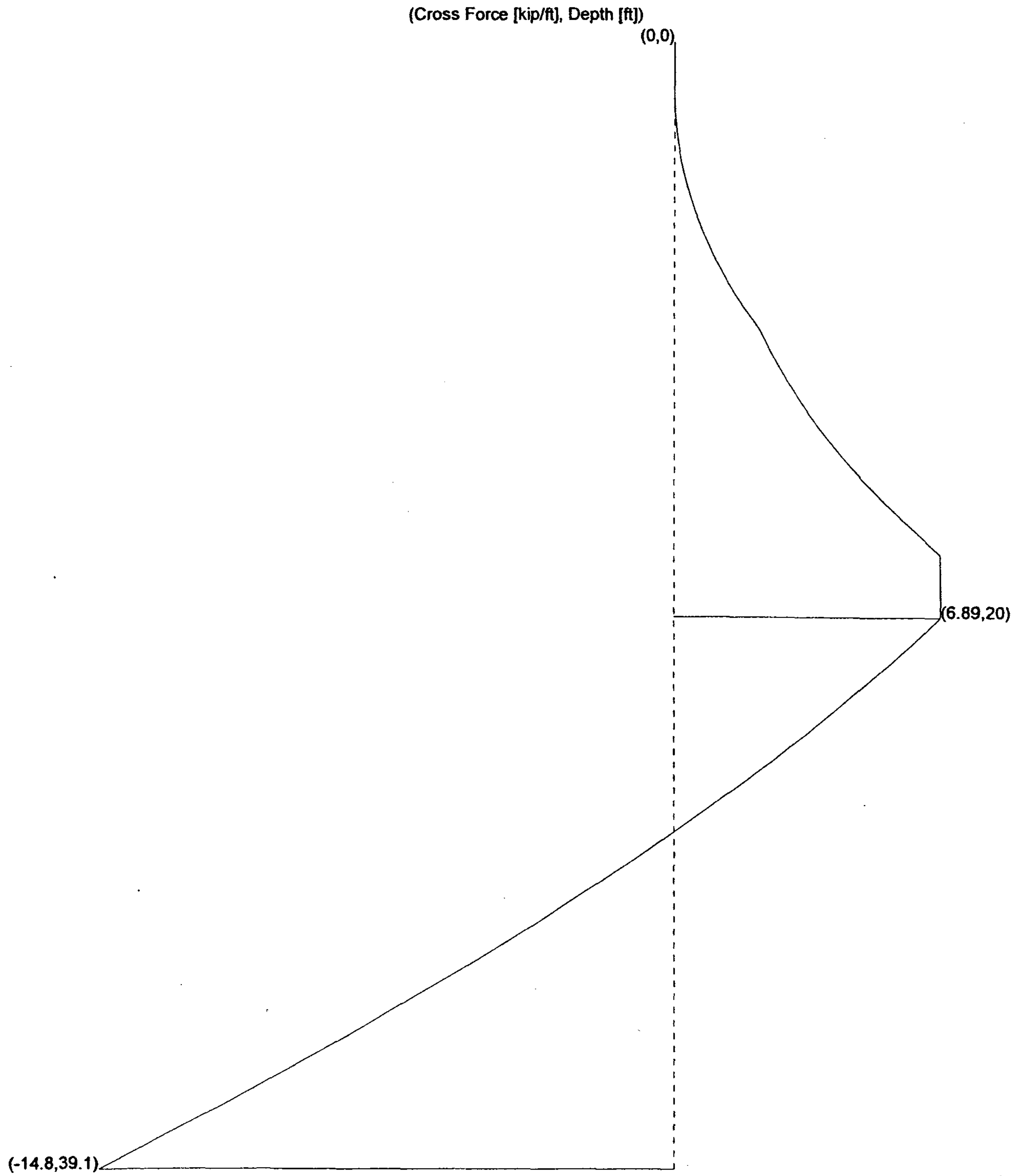
MOMENT DIAGRAM



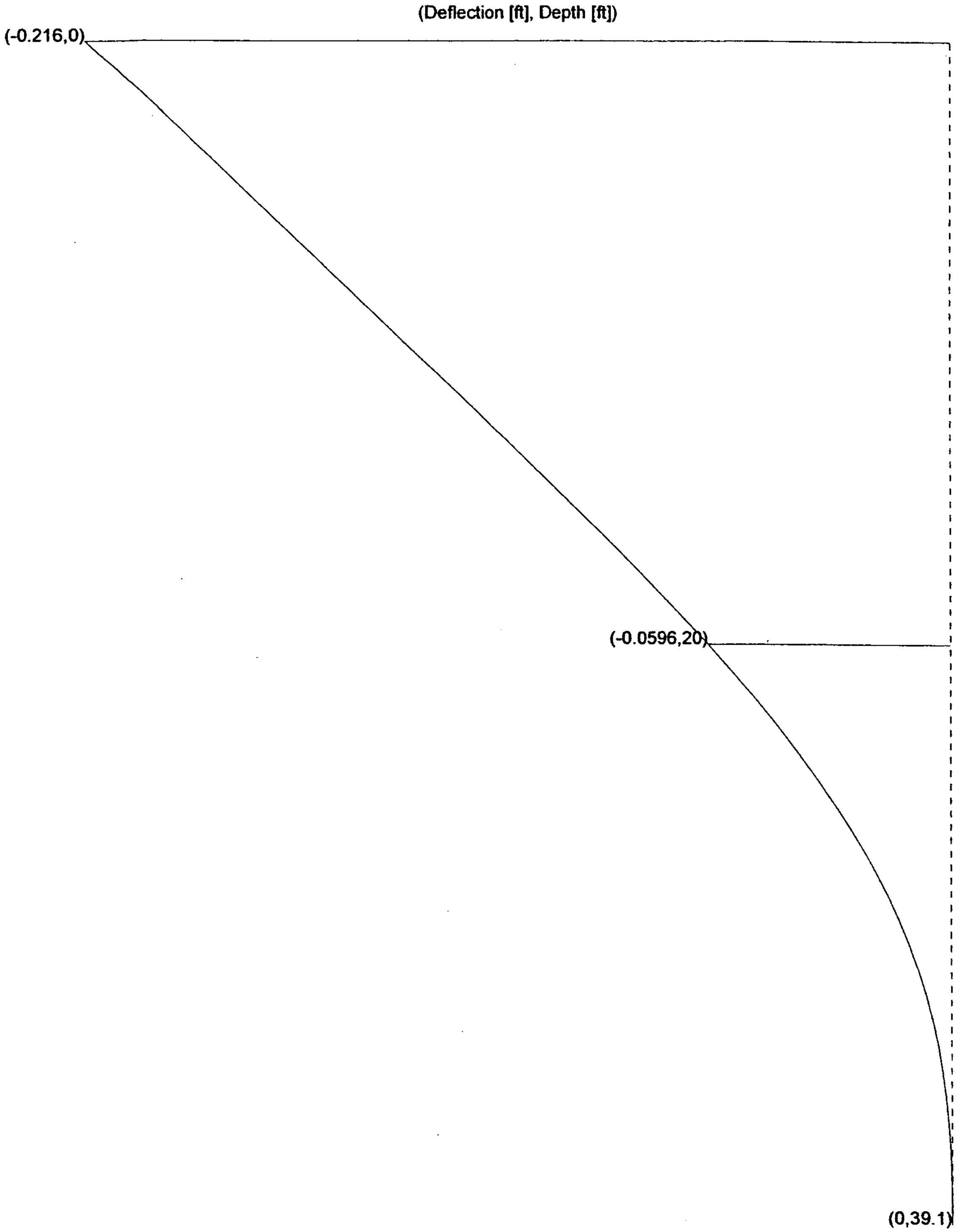
ROTATION DIAGRAM

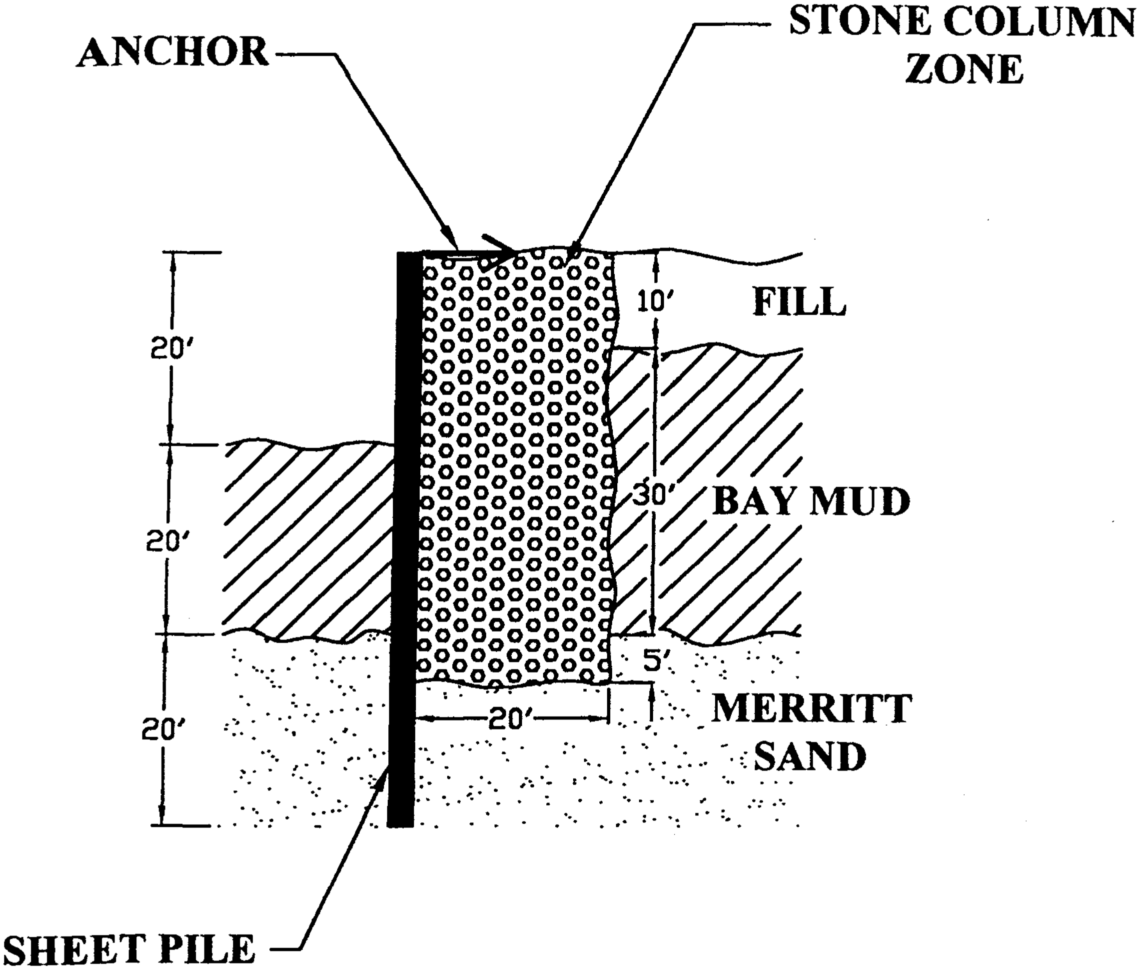


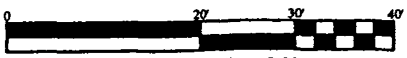
CROSS FORCE DIAGRAM



DEFLECTION DIAGRAM







SCALE: 1" = 20'

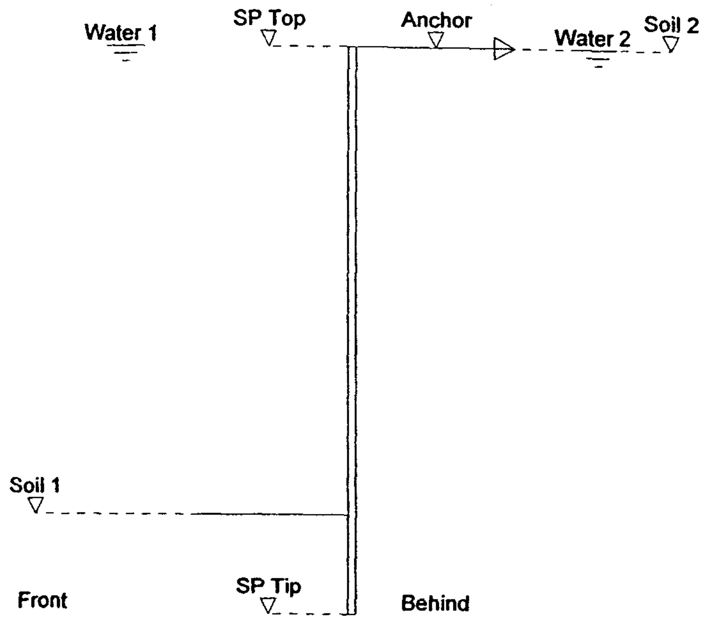
SECTION D-D' - STONE COLUMN AND SHEET PILE
-ANCHORED-

SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/15/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\DDStatic_sc_a.spc
Comment:	Section D-D' Post EQ Soil Properties 20 ft Stone Column Zone Anchored

GEODATA

Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	24.367
Soil Level in Front [ft]	20.000
Soil Level behind [ft]	0.000
Anchorlevel [ft]	0.000
Water Level in Front [ft]	0.000
Water Level behind [ft]	0.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	0.000
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Free



LAYERS IN FRONT

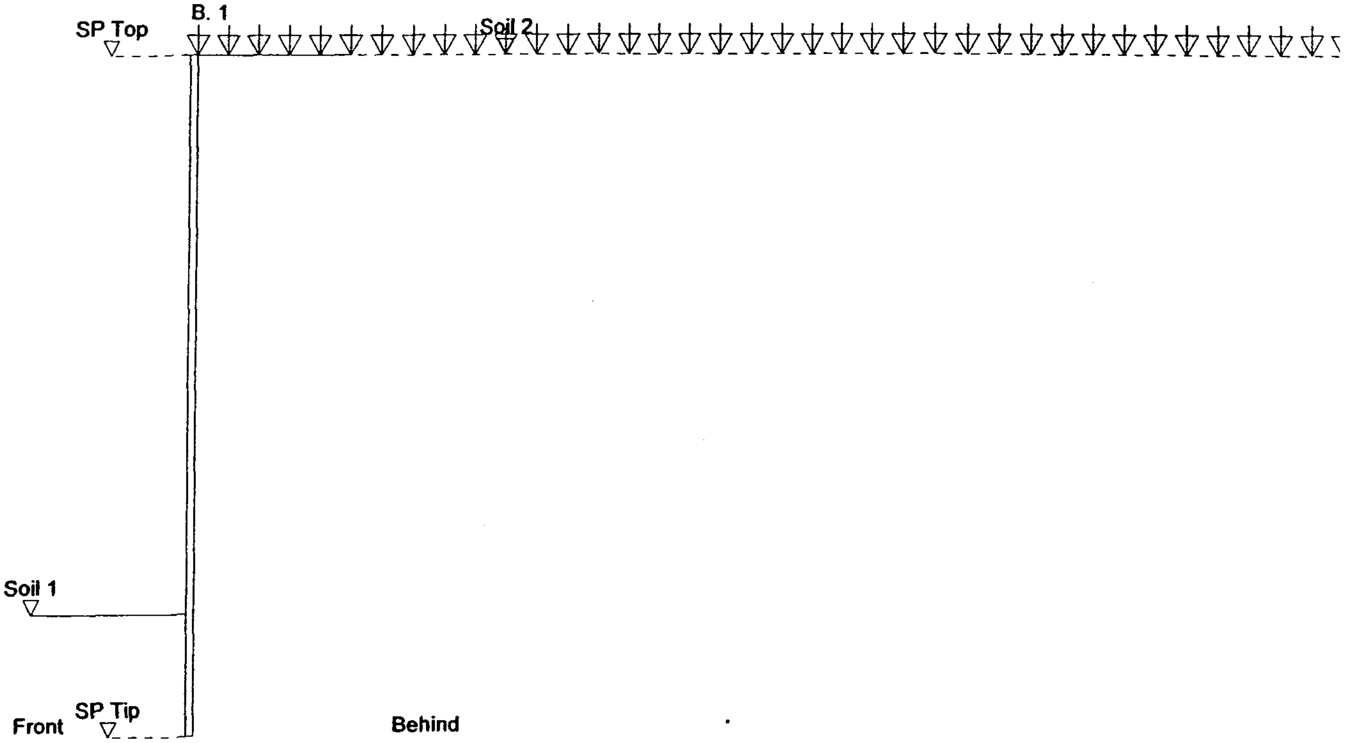
	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2
Layer 1	40.000	0.115	0.052	1.000	0.000	0.000	0.400
Layer 2	120.000	0.130	0.067	4.208	38.000	0.000	0.000

LAYERS BEHIND

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kah	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	10.000	0.128	0.066	1.000	0.000	0.000	0.300
Layer 2	17.800	0.126	0.066	1.000	0.000	0.000	0.400
Layer 3	40.000	0.130	0.068	0.217	40.000	0.000	0.400
Layer 4	45.000	0.130	0.680	0.217	40.000	0.000	0.000
Layer 5	120.000	0.130	0.680	0.238	38.000	0.000	0.000

BOUSSINESQ

	Distance Wall [ft]	Width Surcharge [ft]	Depth Surcharge [ft]	Surcharge [kip/ft2]
Bousq. 1	0.000	300.000	0.000	0.480



6

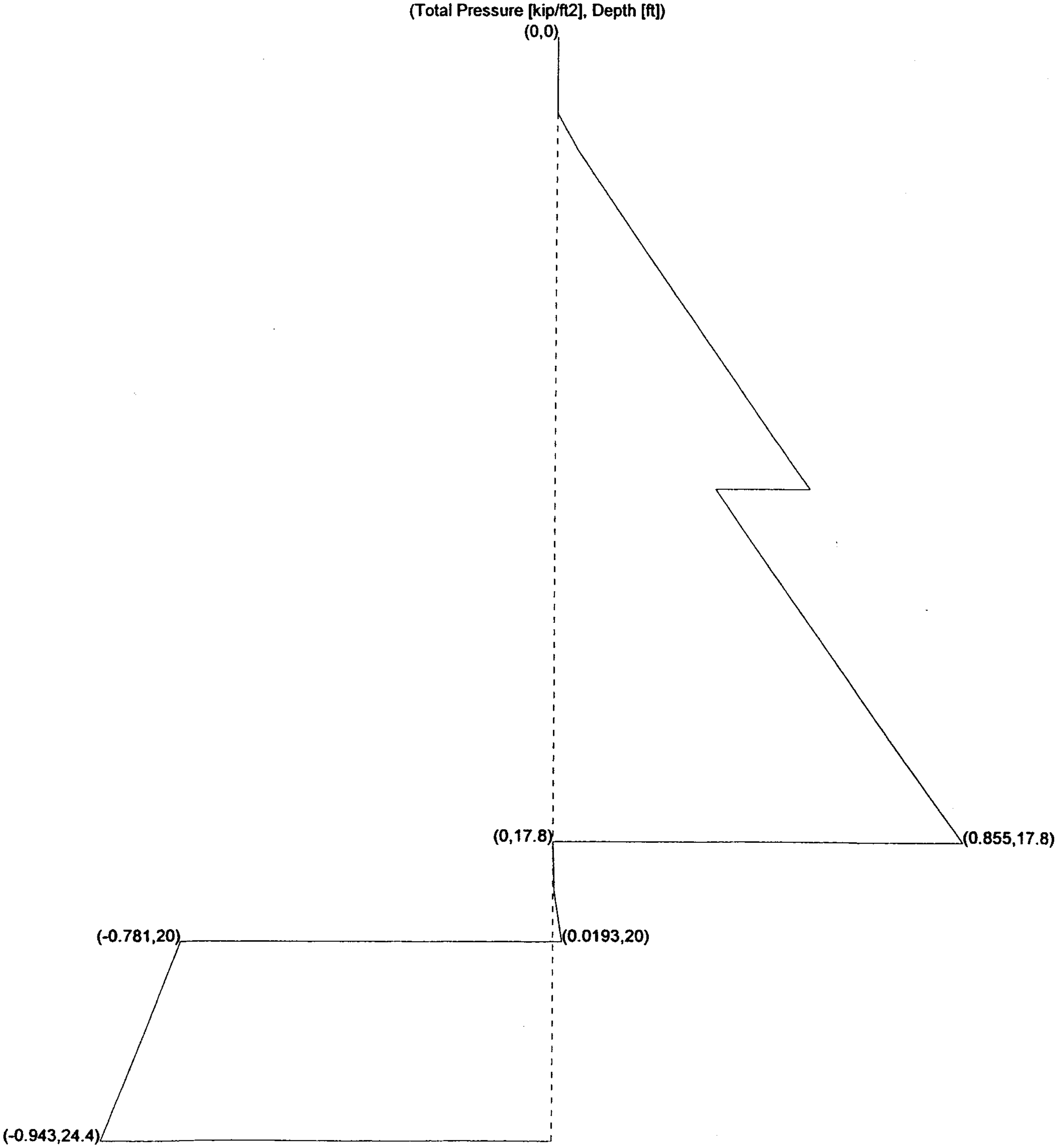
PILE SECTION

Name	AZ48
Inertia [in4/ft]	847.024
Modulus [in3/ft]	89.280
Area [in2/ft]	14.481
Mass [lbs/ft2]	49.279
Steelgrade [lb/in2]	60000.003
Requested Safety	2.000

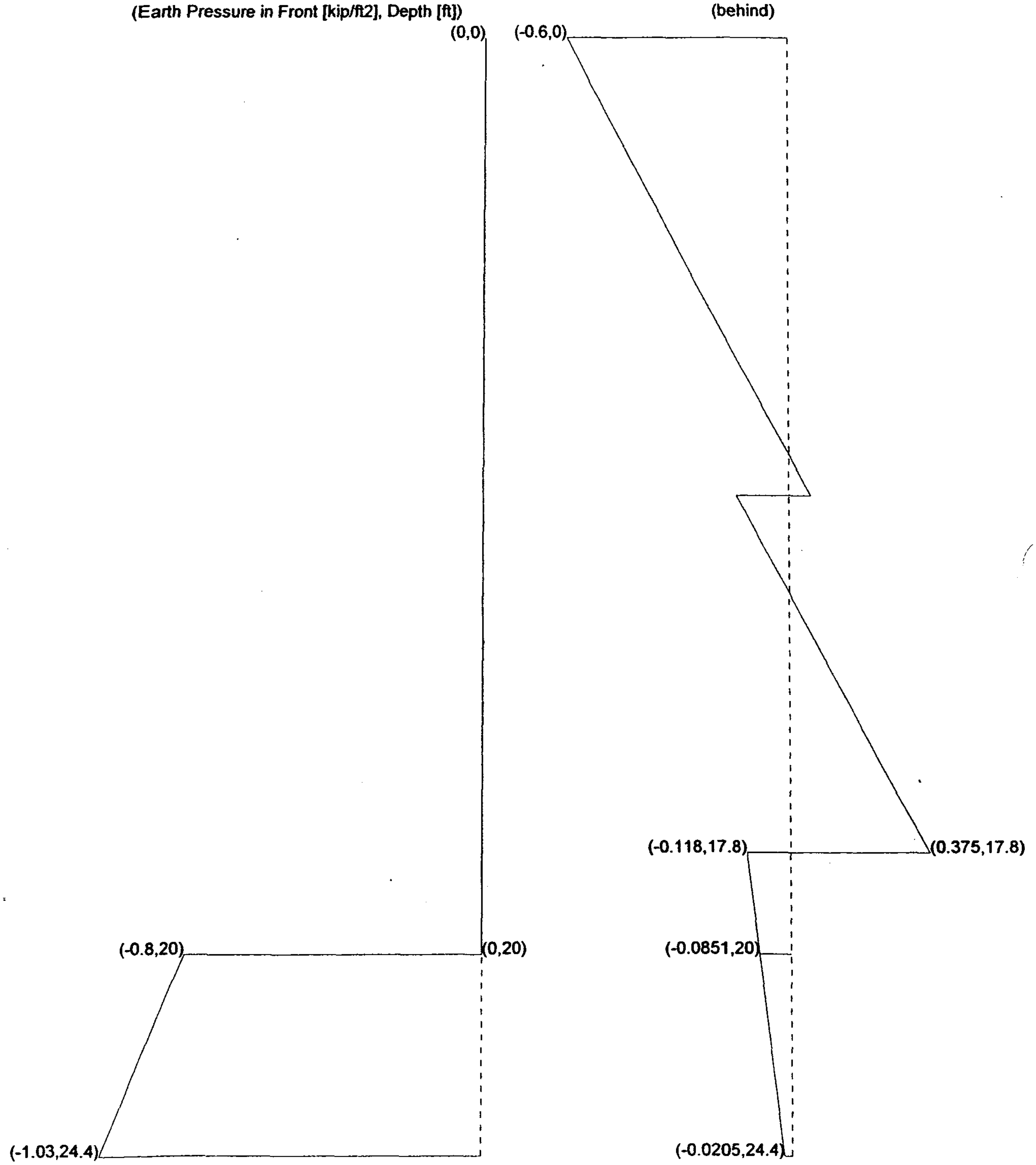
PILE CHECK

		Depth [ft]
Chosen Sheet Pile Section	AZ48	
Moment of Inertia [in4/ft]	847.024	
Section Modulus [in3/ft]	89.280	
Area [in2/ft]	14.481	
Mass [lbs/ft2]	49.279	
Steel Grade [lb/in2]	60000.003	
Minimal Moment [kipft/ft]	-26.245	12.225
Maximal Moment [kipft/ft]	-0.000	24.396
Normal Forces at Min. Moment [kip/ft]	0.000	12.225
Normal Forces at Max. Moment [kip/ft]	0.000	24.396
Deflection at Min. Moment [ft]	-0.008	12.225
Deflection at Max. Moment [ft]	0.000	24.396
Min. Stress at Min. Moment [lb/in2]	-3527.484	12.225
Max. Stress at Min. Moment [lb/in2]	3527.484	12.225
Min. Stress at Max. Moment [lb/in2]	-0.061	24.396
Max. Stress at Max. Moment [lb/in2]	0.061	24.396
Safety > Req. Safety = 2.000	17.009	
Pile Top [ft]		0.000
Pile Tip [ft]		24.367
Vertical Equilibrium [kip/ft]	0.000	
Anchor Force (horiz.) [kip/ft]	-3.123	0.000

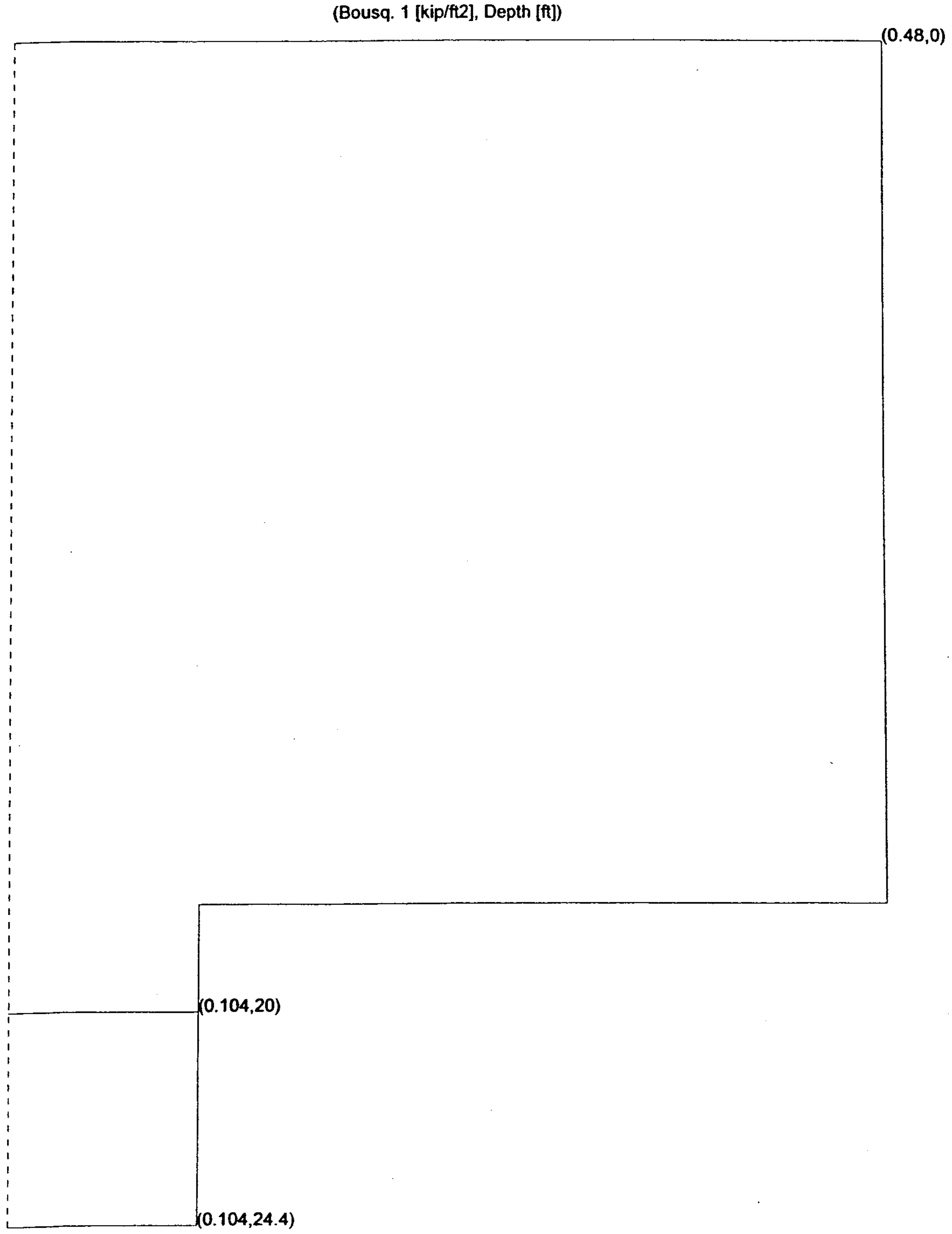
TOTAL PRESSURE DIAGRAM



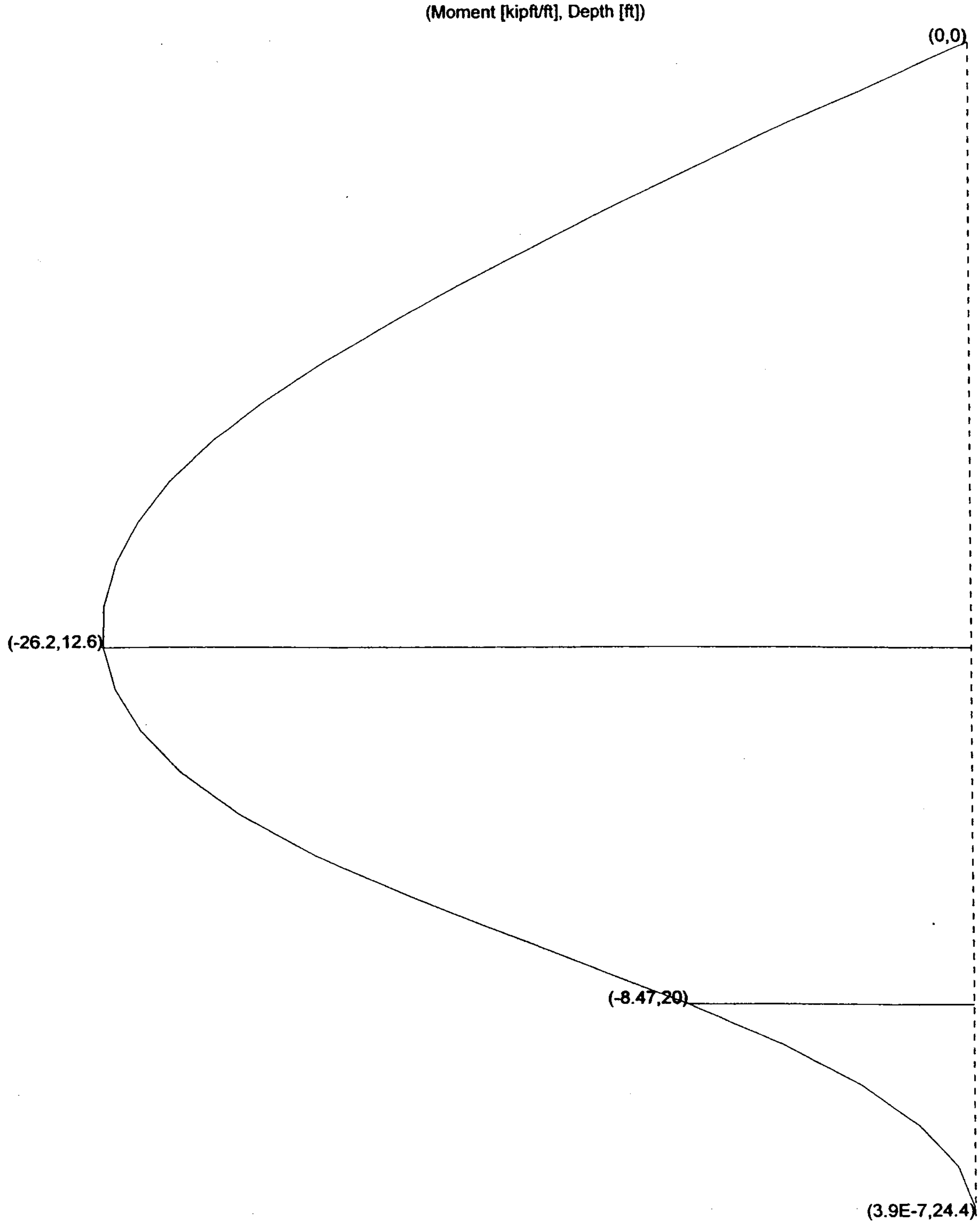
EARTH PRESSURE DIAGRAM



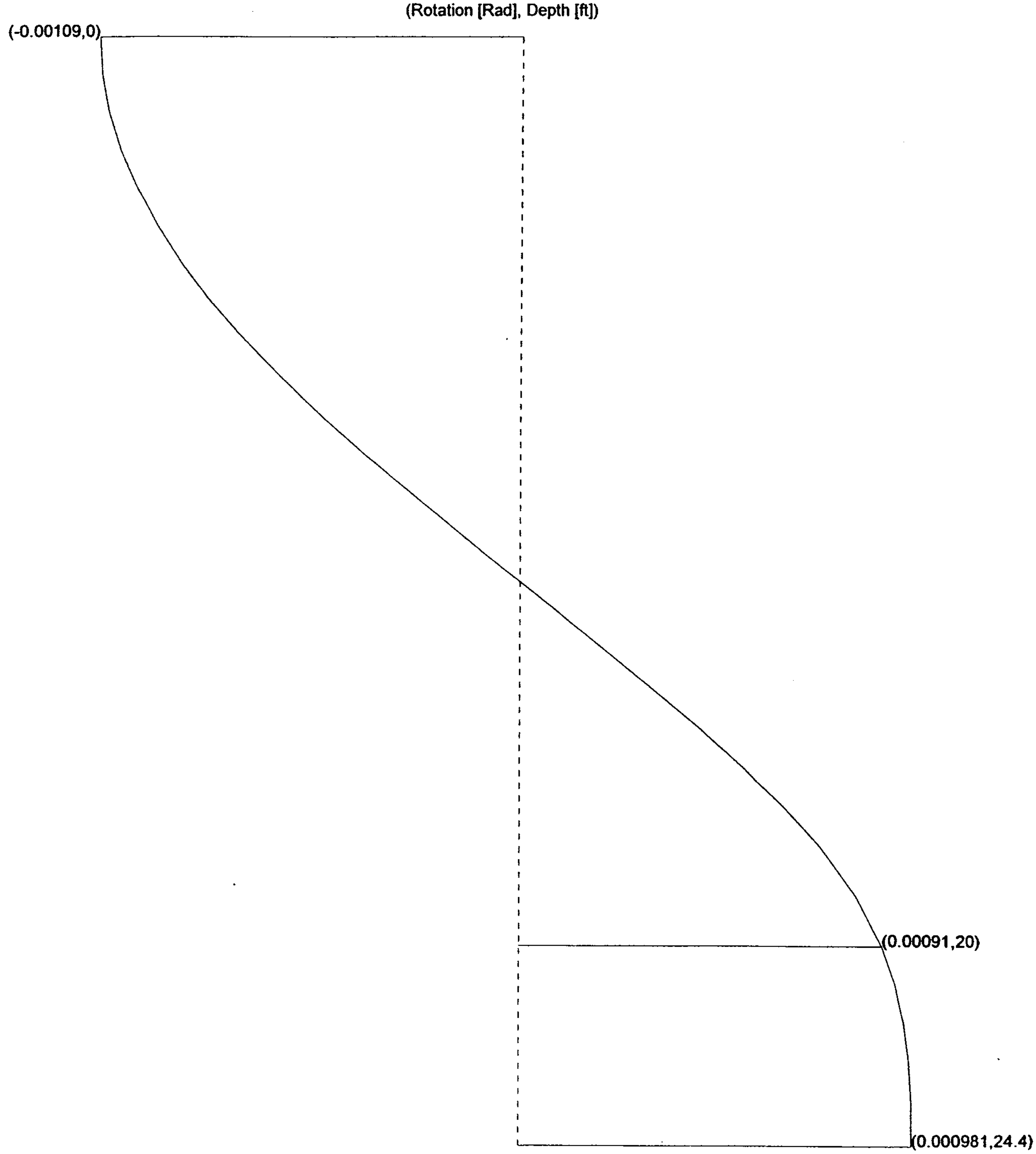
BOUSSINESQ DIAGRAM



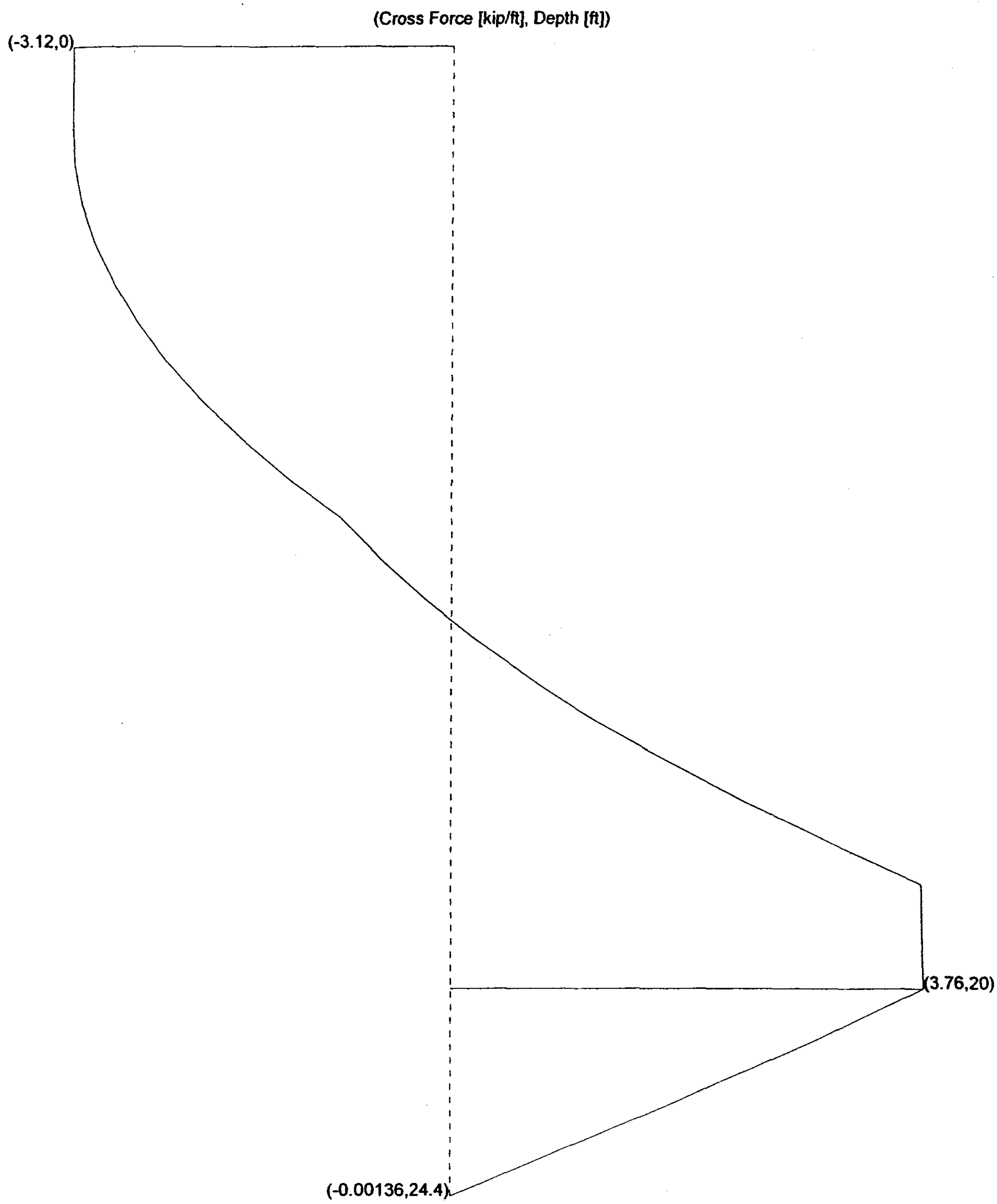
MOMENT DIAGRAM



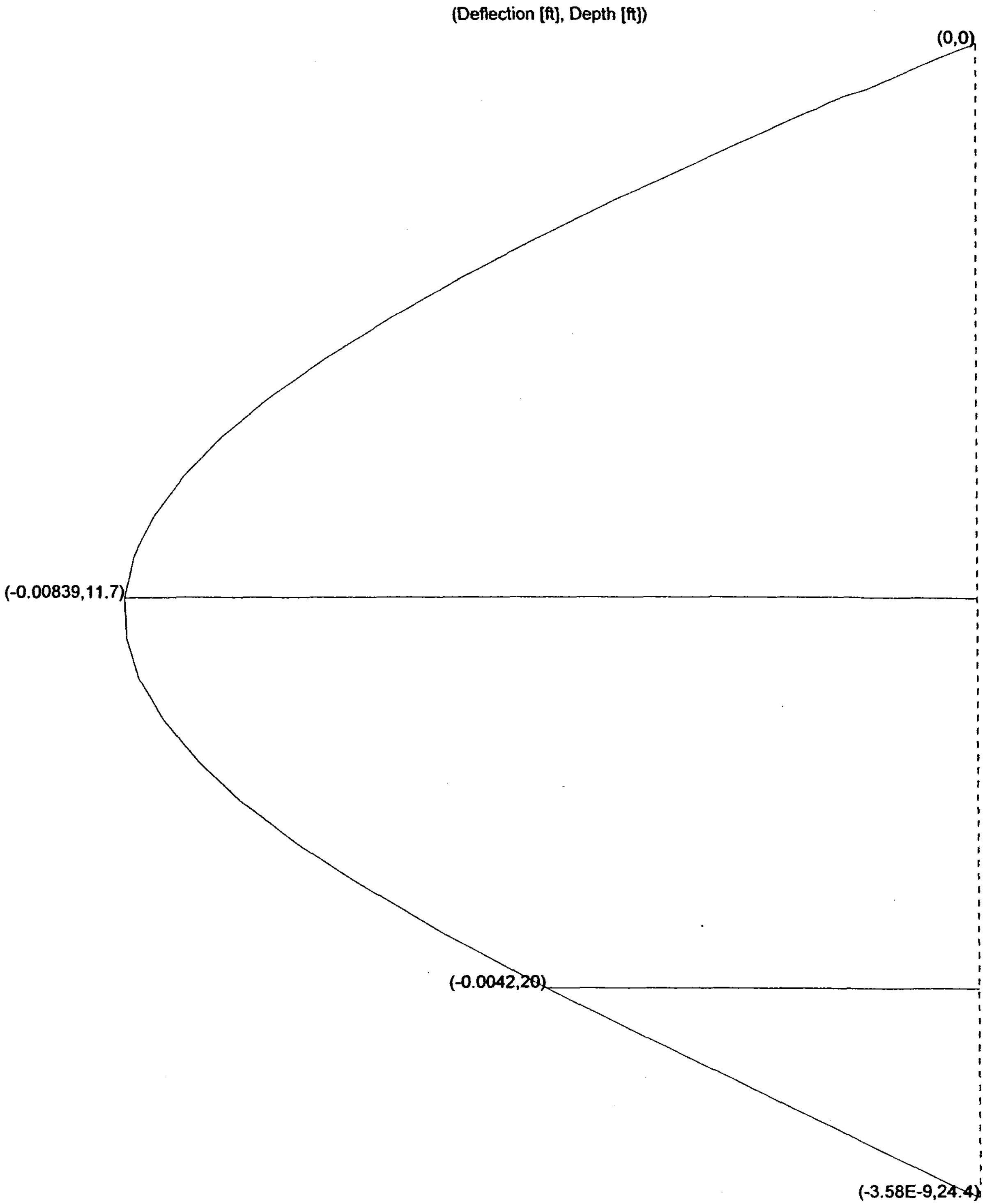
ROTATION DIAGRAM



CROSS FORCE DIAGRAM



DEFLECTION DIAGRAM

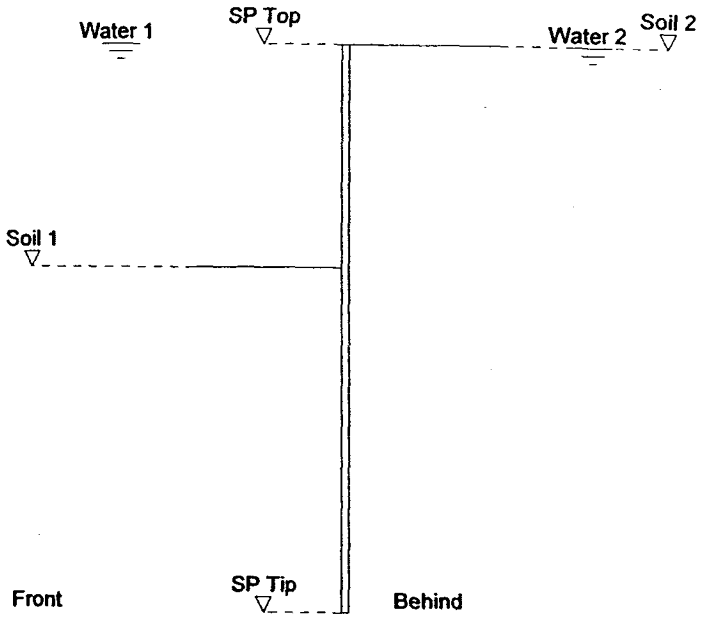


SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/15/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\DSeismic_sc_a5.spc
Comment:	Section D-D' Avg. Pre- and Post-EQ Soil Prop. 20 ft Stone Column Zone 5 kip Anchor

GEODATA

Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	50.581
Soil Level in Front [ft]	20.000
Soil Level behind [ft]	0.000
Anchorlevel [ft]	0.000
Water Level in Front [ft]	0.000
Water Level behind [ft]	0.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	0.000
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Cantilever



LAYERS IN FRONT

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	40.000	0.115	0.052	1.000	0.000	0.000	0.450
Layer 2	120.000	0.130	0.067	4.208	38.000	0.000	0.000

LAYERS BEHIND

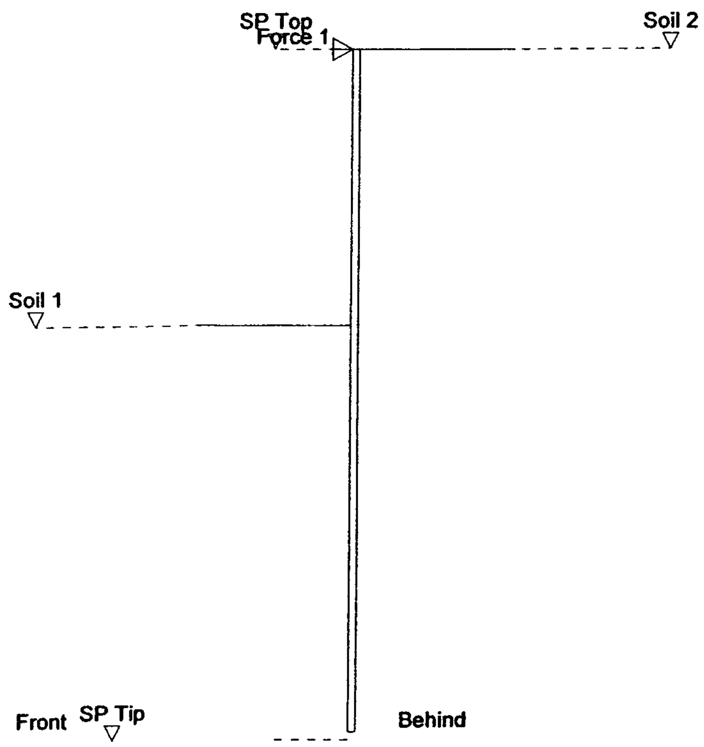
	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kah	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	10.000	0.128	0.066	0.548	17.000	0.000	0.200
Layer 2	17.800	0.126	0.066	1.000	0.000	0.000	0.450
Layer 3	40.000	0.130	0.068	0.217	40.000	0.000	0.450
Layer 4	45.000	0.130	0.680	0.217	40.000	0.000	0.000
Layer 5	120.000	0.130	0.680	0.238	38.000	0.000	0.000

USERDEFINED PRESSURES

	Pressure Top [kip/ft2]	Pressure Tip [kip/ft2]	Depth Top [ft]	Depth Tip [ft]
Strip 1	0.600	0.600	0.000	60.000

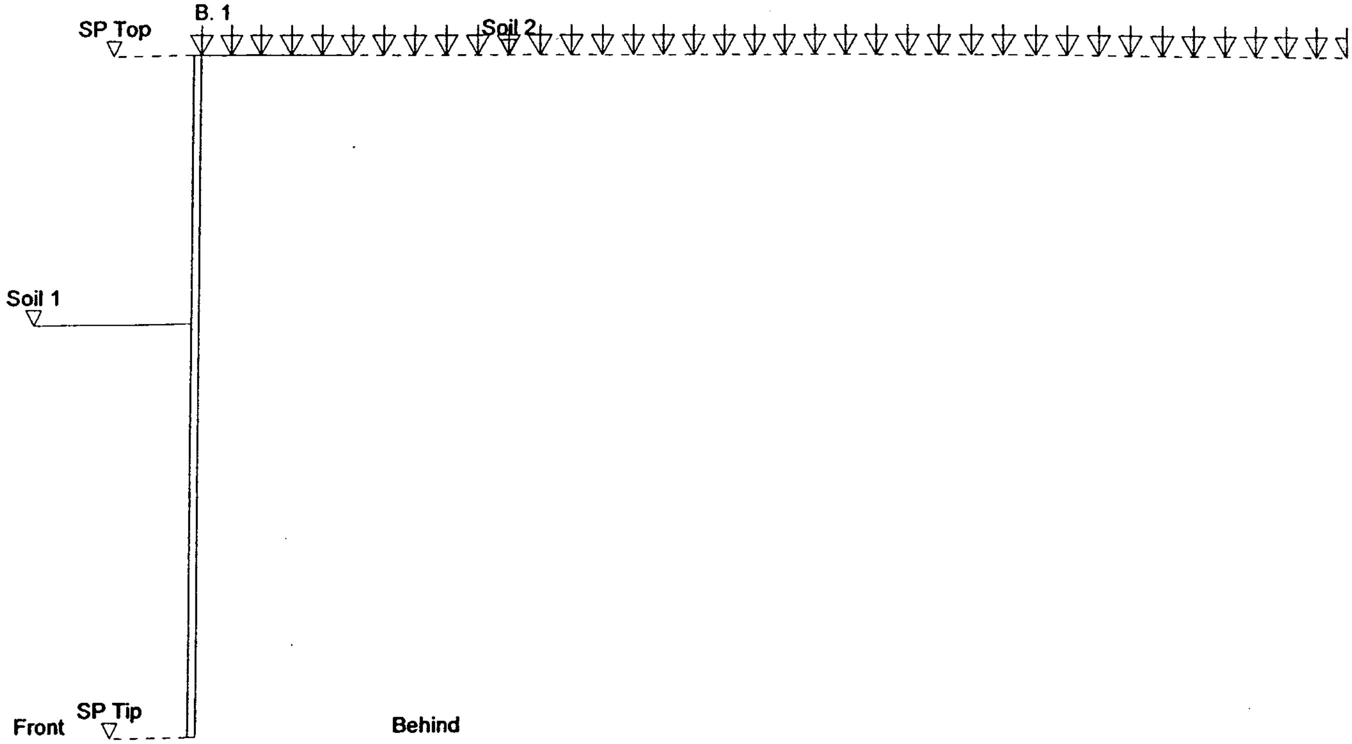
CONCENTRATED FORCES

	Horiz. Component [kip/ft]	Vert. Component [kip/ft]	Depth Horiz. Comp. [ft]
Force 1	-5.000	0.000	0.000



BOUSSINESQ

	Distance Wall [ft]	Width Surcharge [ft]	Depth Surcharge [ft]	Surcharge [kip/ft2]
Bousq. 1	0.000	300.000	0.000	0.480



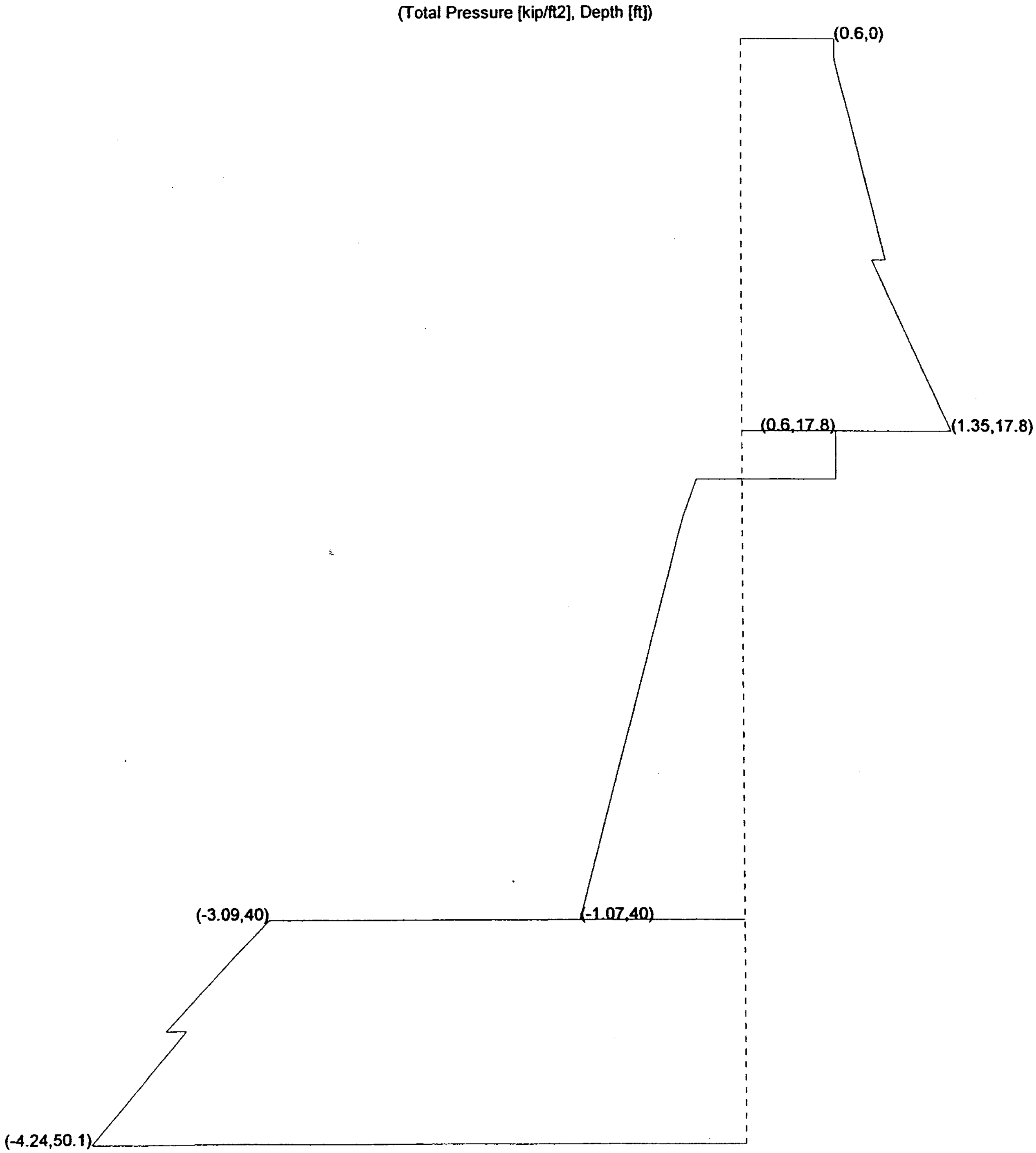
PILE SECTION

Name	AZ48
Inertia [in4/ft]	847.024
Modulus [in3/ft]	89.280
Area [in2/ft]	14.481
Mass [lbs/ft2]	49.279
Steelgrade [lb/in2]	60000.003
Requested Safety	2.000

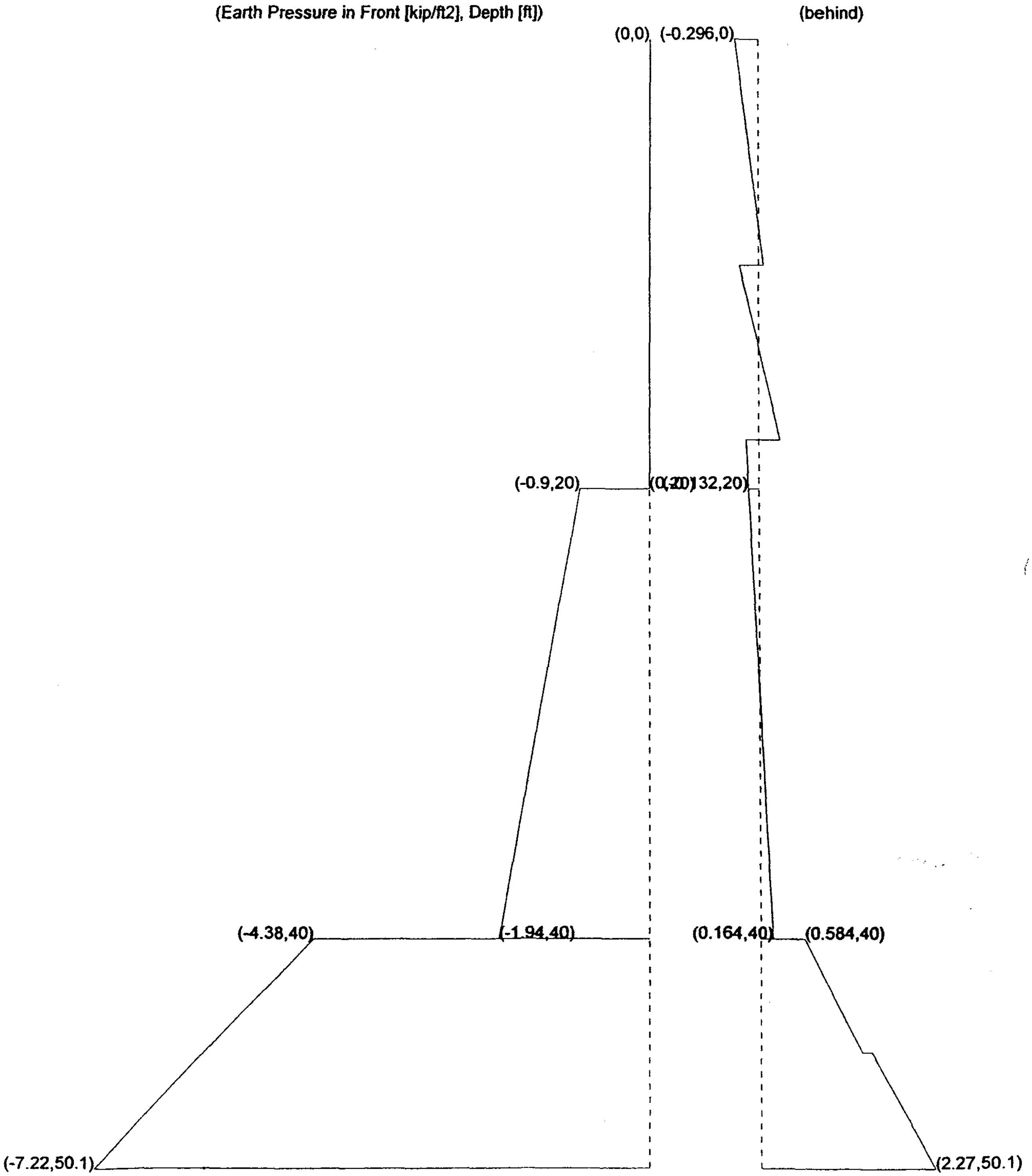
PILE CHECK

		Depth [ft]
Chosen Sheet Pile Section	AZ48	
Moment of Inertia [in4/ft]	847.024	
Section Modulus [in3/ft]	89.280	
Area [in2/ft]	14.481	
Mass [lbs/ft2]	49.279	
Steel Grade [lb/in2]	60000.003	
Minimal Moment [kipft/ft]	-18.949	7.159
Maximal Moment [kipft/ft]	194.525	38.353
Normal Forces at Min. Moment [kip/ft]	0.000	7.159
Normal Forces at Max. Moment [kip/ft]	0.000	38.353
Deflection at Min. Moment [ft]	-0.693	7.159
Deflection at Max. Moment [ft]	-0.040	38.353
Min. Stress at Min. Moment [lb/in2]	-2546.761	7.159
Max. Stress at Min. Moment [lb/in2]	2546.761	7.159
Min. Stress at Max. Moment [lb/in2]	26144.762	38.353
Max. Stress at Max. Moment [lb/in2]	26144.762	38.353
Safety > Req. Safety = 2.000	2.295	
Pile Top [ft]		0.000
Pile Tip [ft]		50.581
Vertical Equilibrium [kip/ft]	0.000	
Anchor Force (horiz.) [kip/ft]	0.000	0.000

TOTAL PRESSURE DIAGRAM

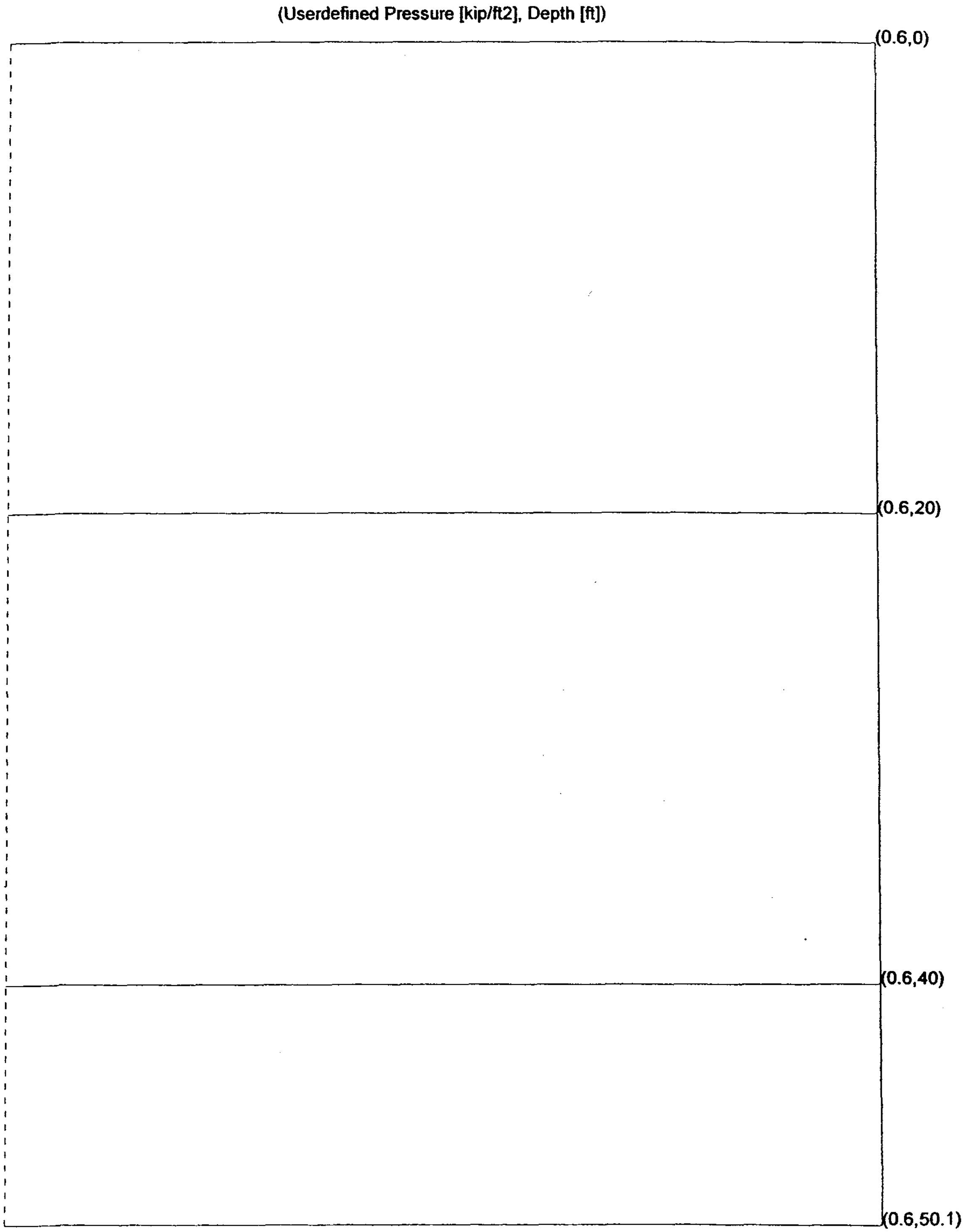


EARTH PRESSURE DIAGRAM

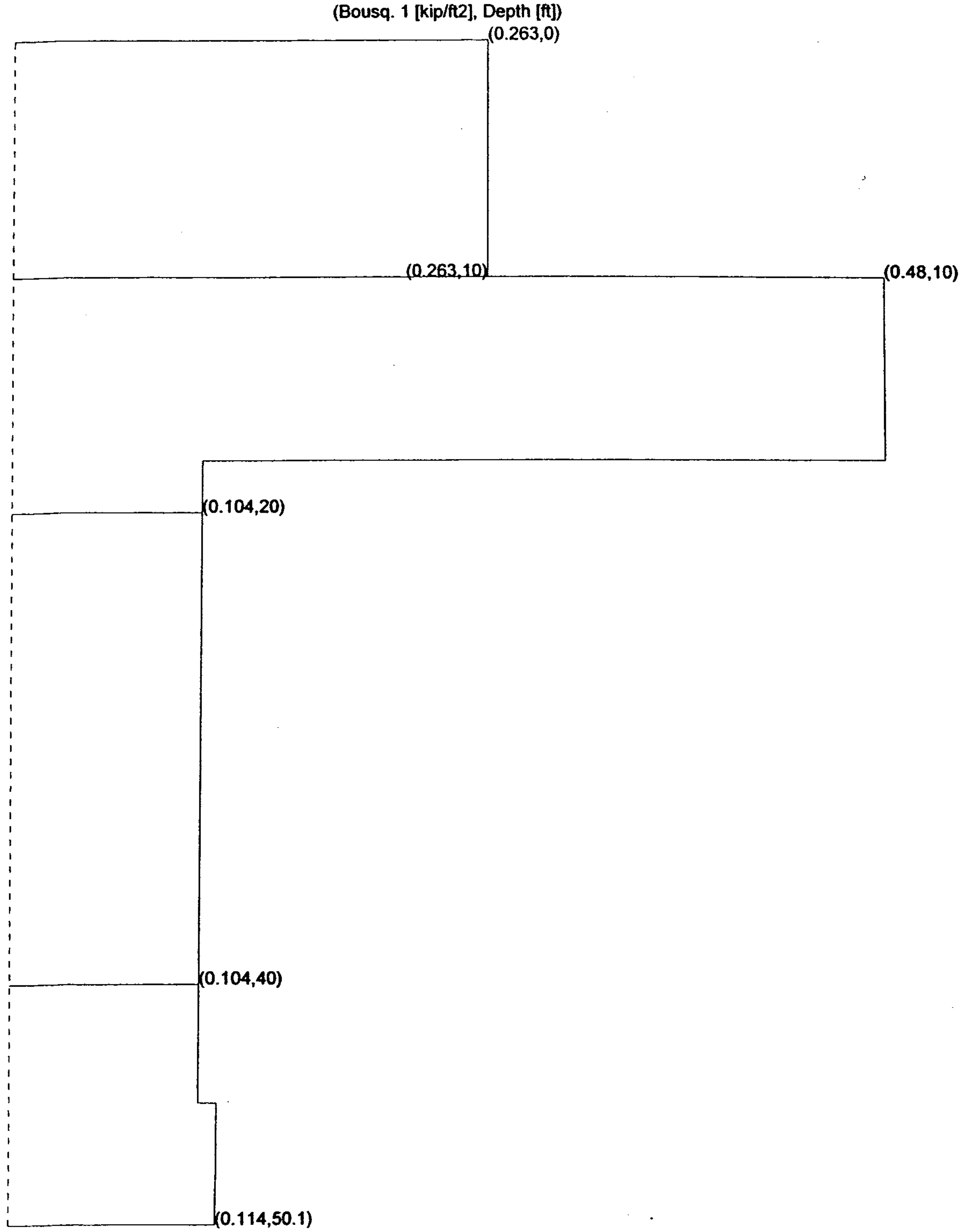


7

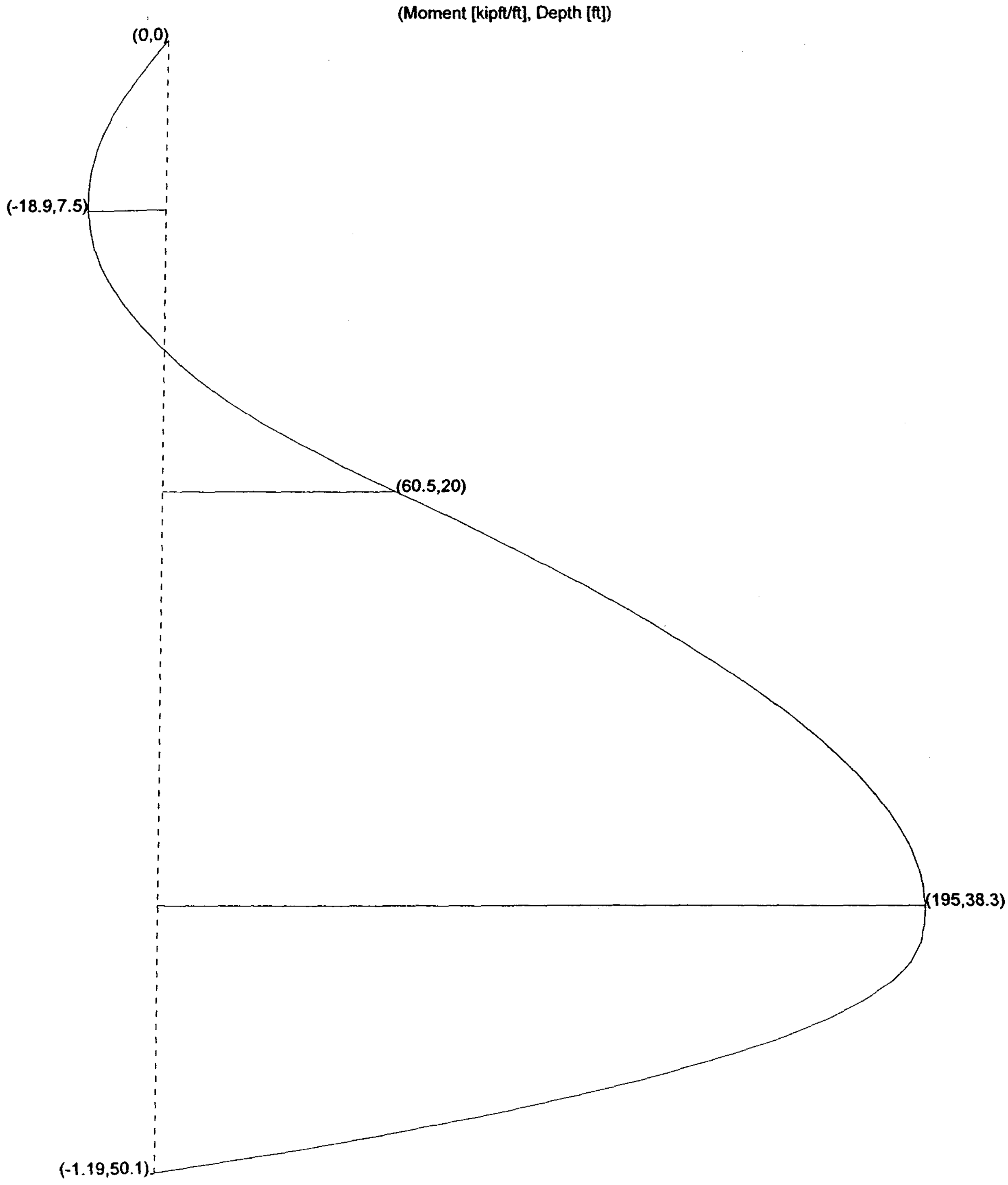
USERDEFINED PRESSURE DIAGRAM



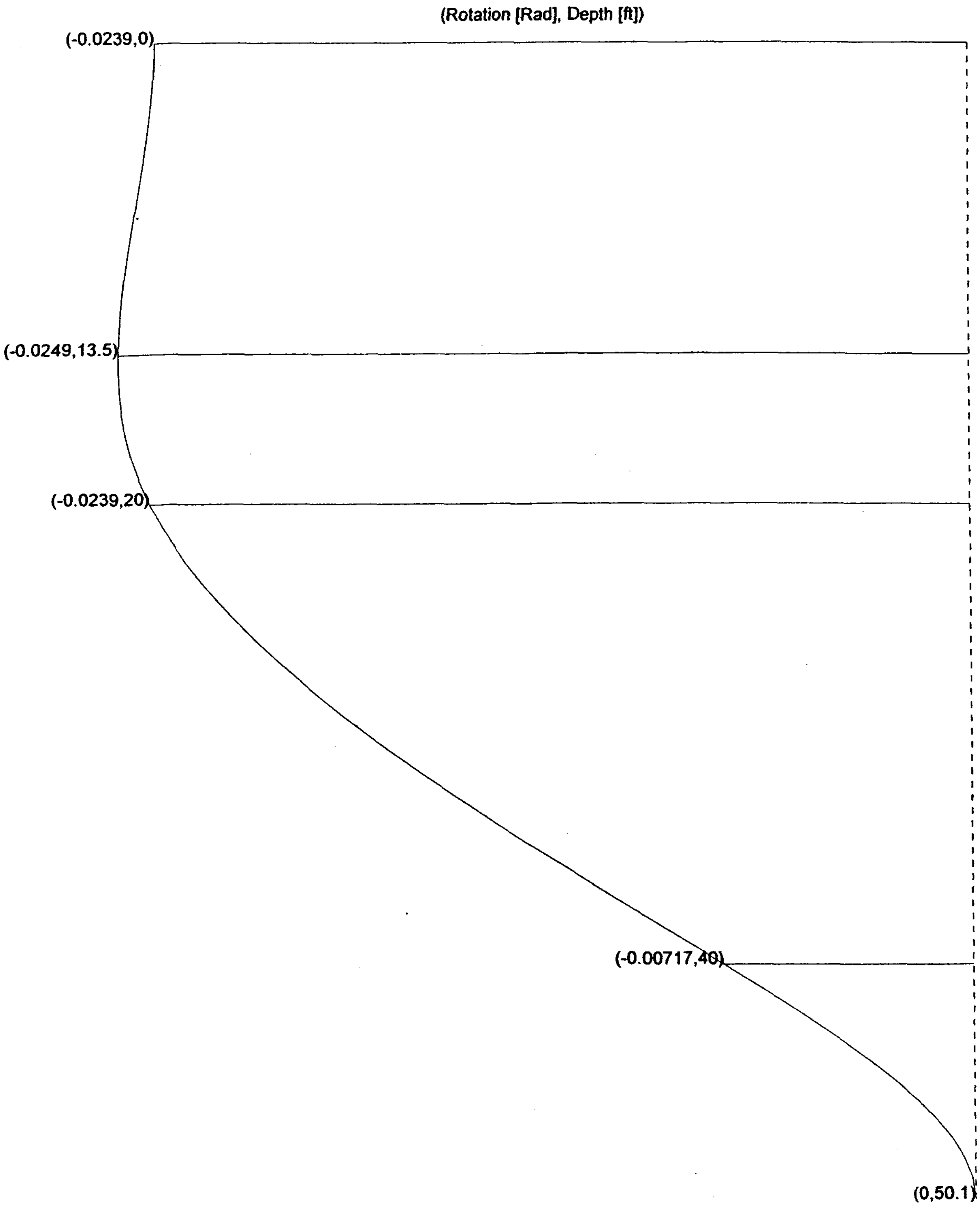
BOUSSINESQ DIAGRAM



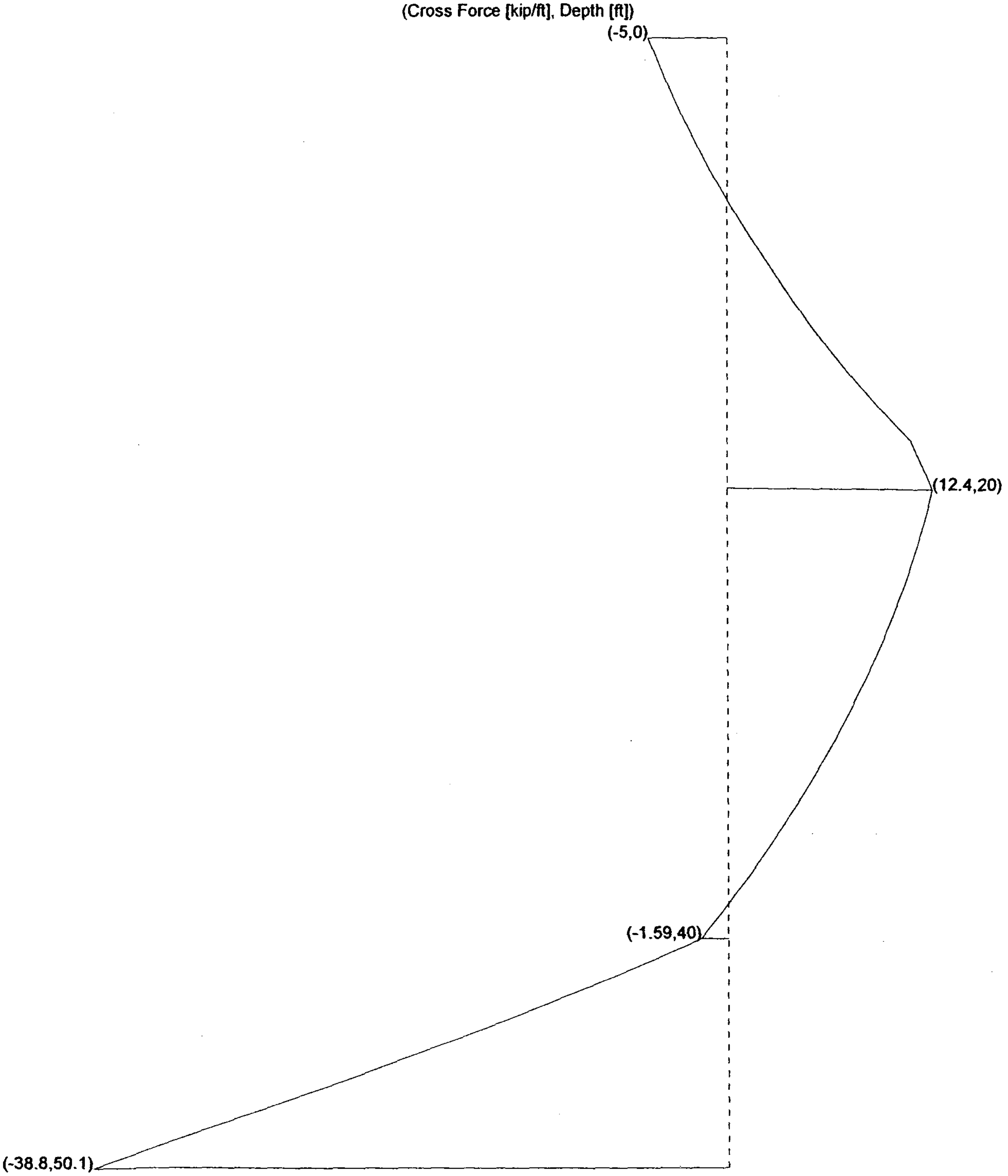
MOMENT DIAGRAM



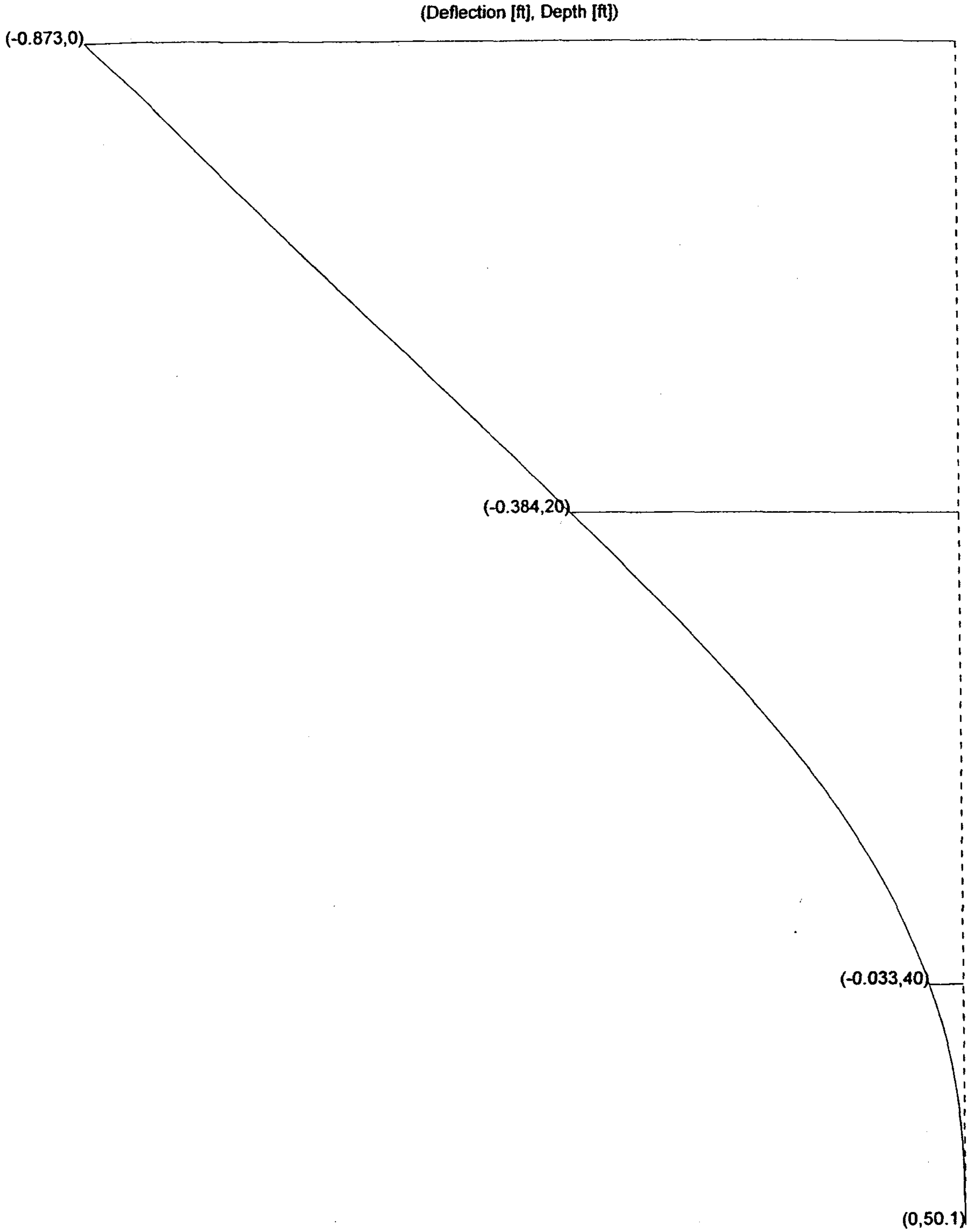
ROTATION DIAGRAM



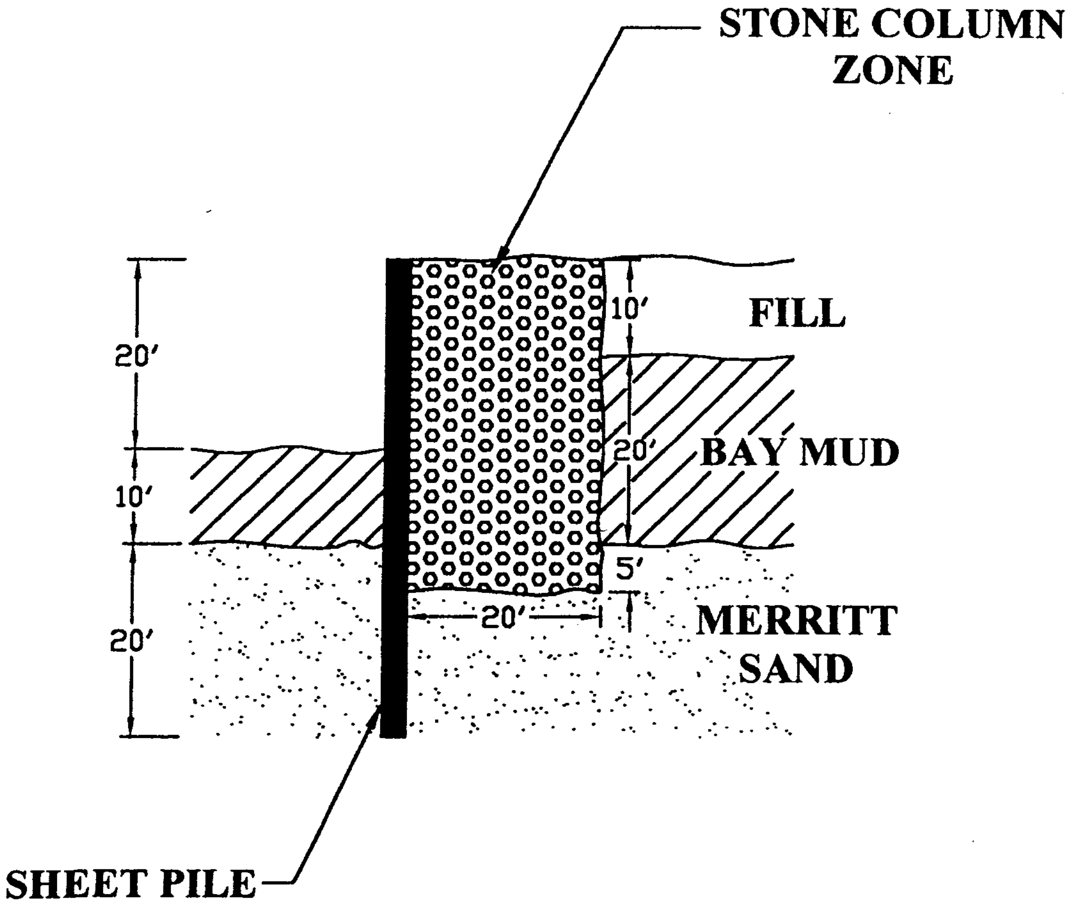
CROSS FORCE DIAGRAM

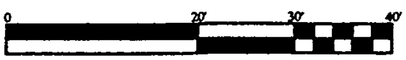


DEFLECTION DIAGRAM



B-





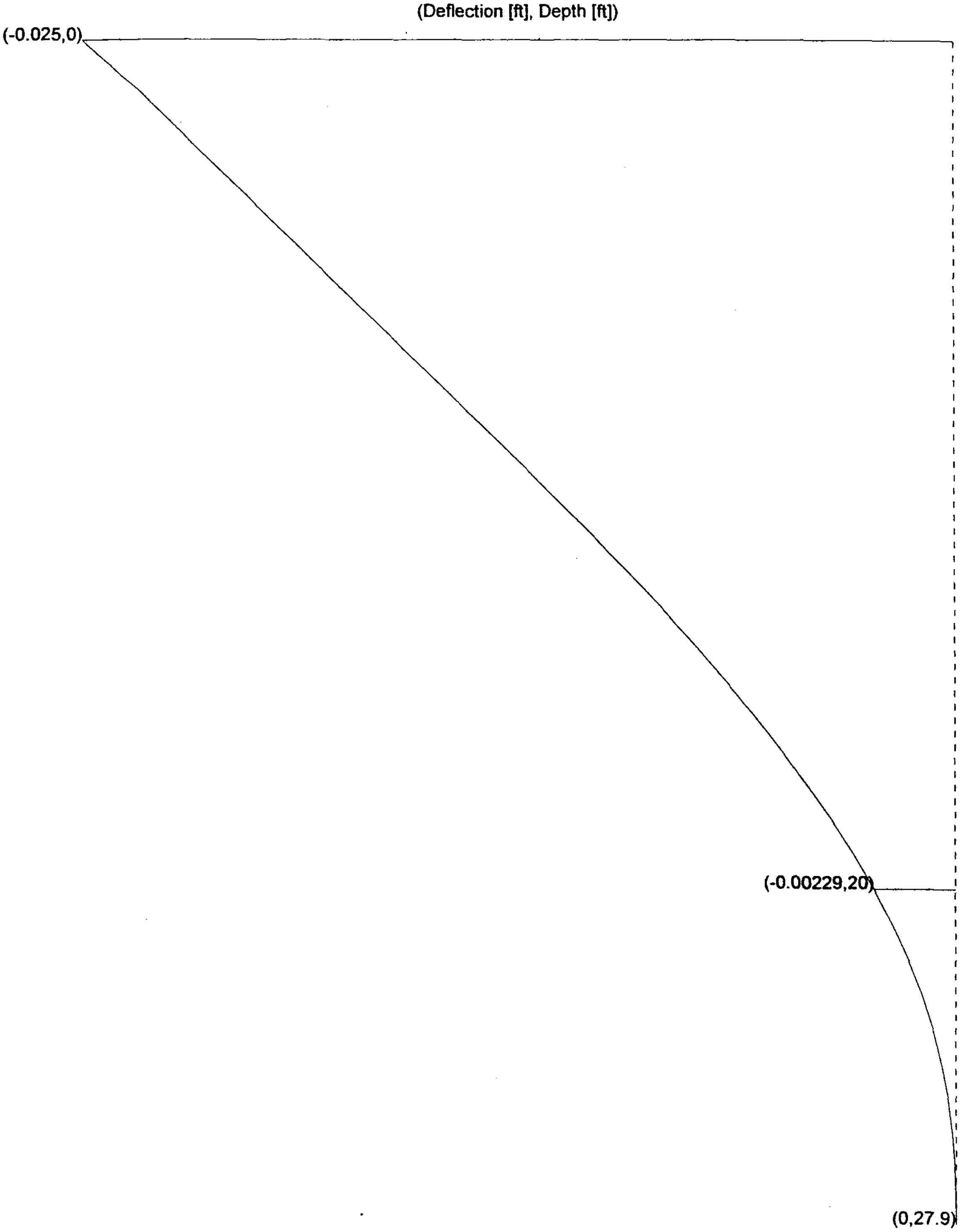
SCALE: 1" = 20'

SECTION F-F' - STONE COLUMN AND SHEET PILE -CANTILIVER-

SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\FStatic_sc.spc
Comment:	Section F-F' Post-EQ Soil Properties 20 ft Stone Column Zone Cantilevered

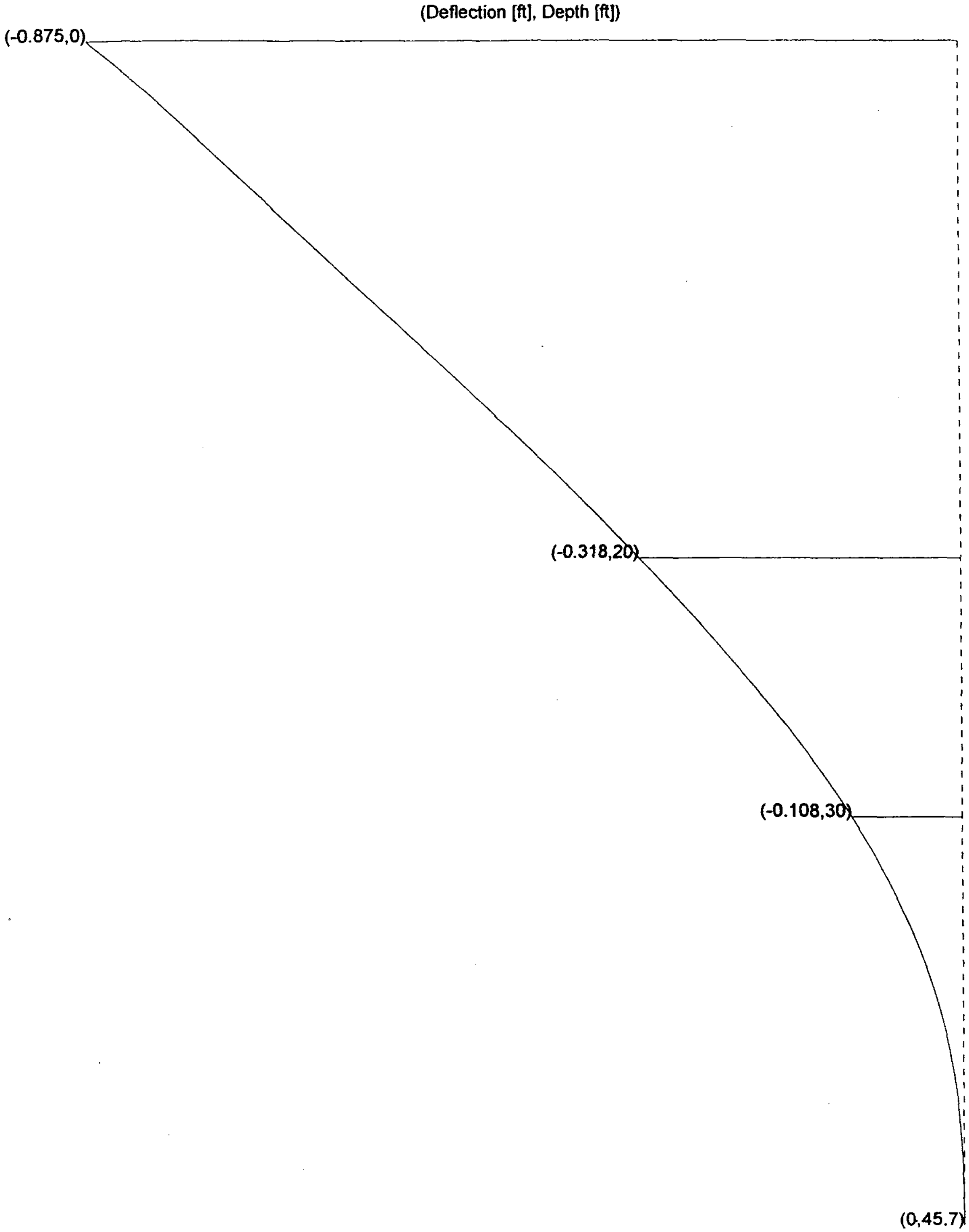
DEFLECTION DIAGRAM

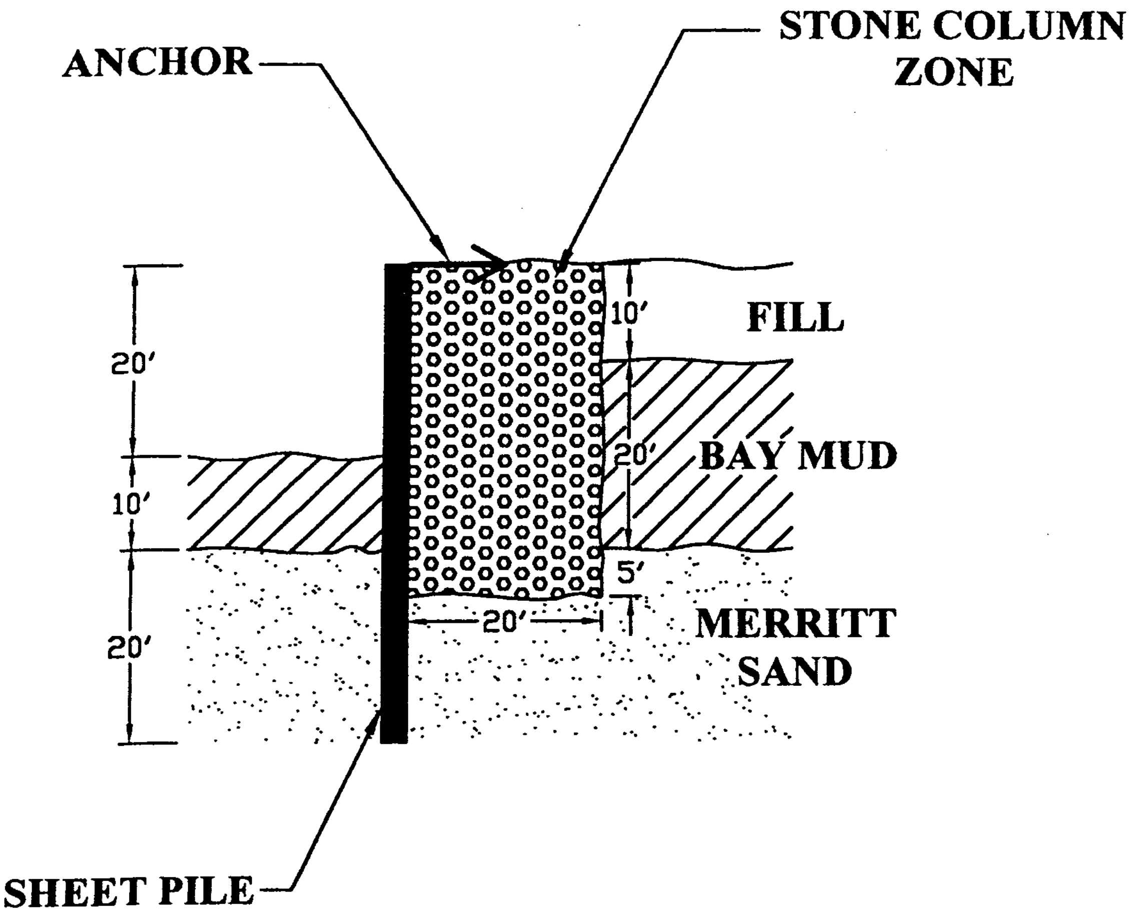


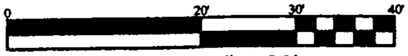
SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\FSeismic_sc.spc
Comment:	Section F-F' Avg'd Pre- and Post-EQ Soil 20 ft Stone Column Zone Cantilevered

DEFLECTION DIAGRAM







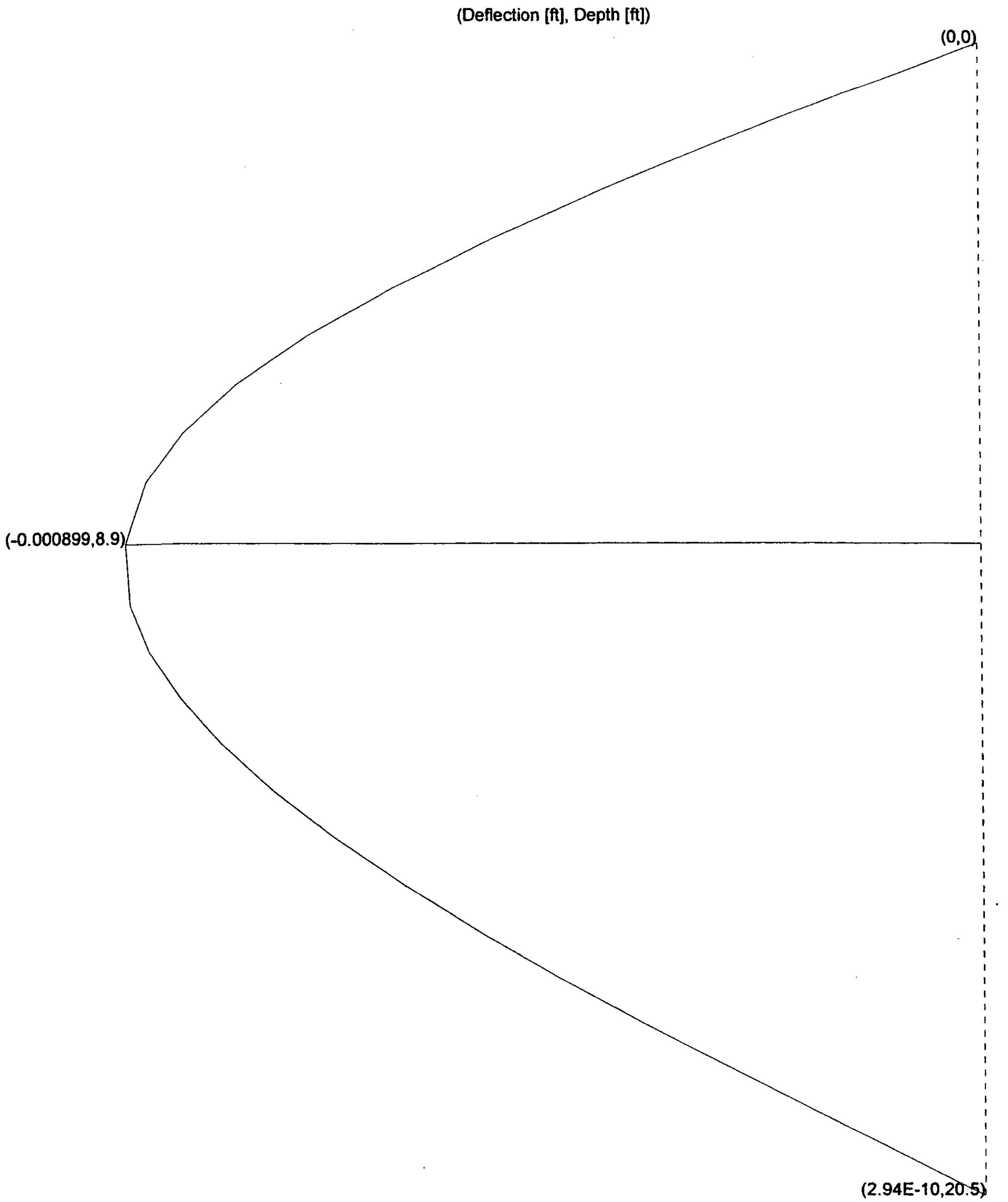
SCALE: 1" = 20'

SECTION F-F' - STONE COLUMN AND SHEET PILE
-ANCHORED-

SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\FStatic_sc_a.spc
Comment:	Section F-F' Post-EQ Soil Properties 20 ft Stone Column Zone Anchored

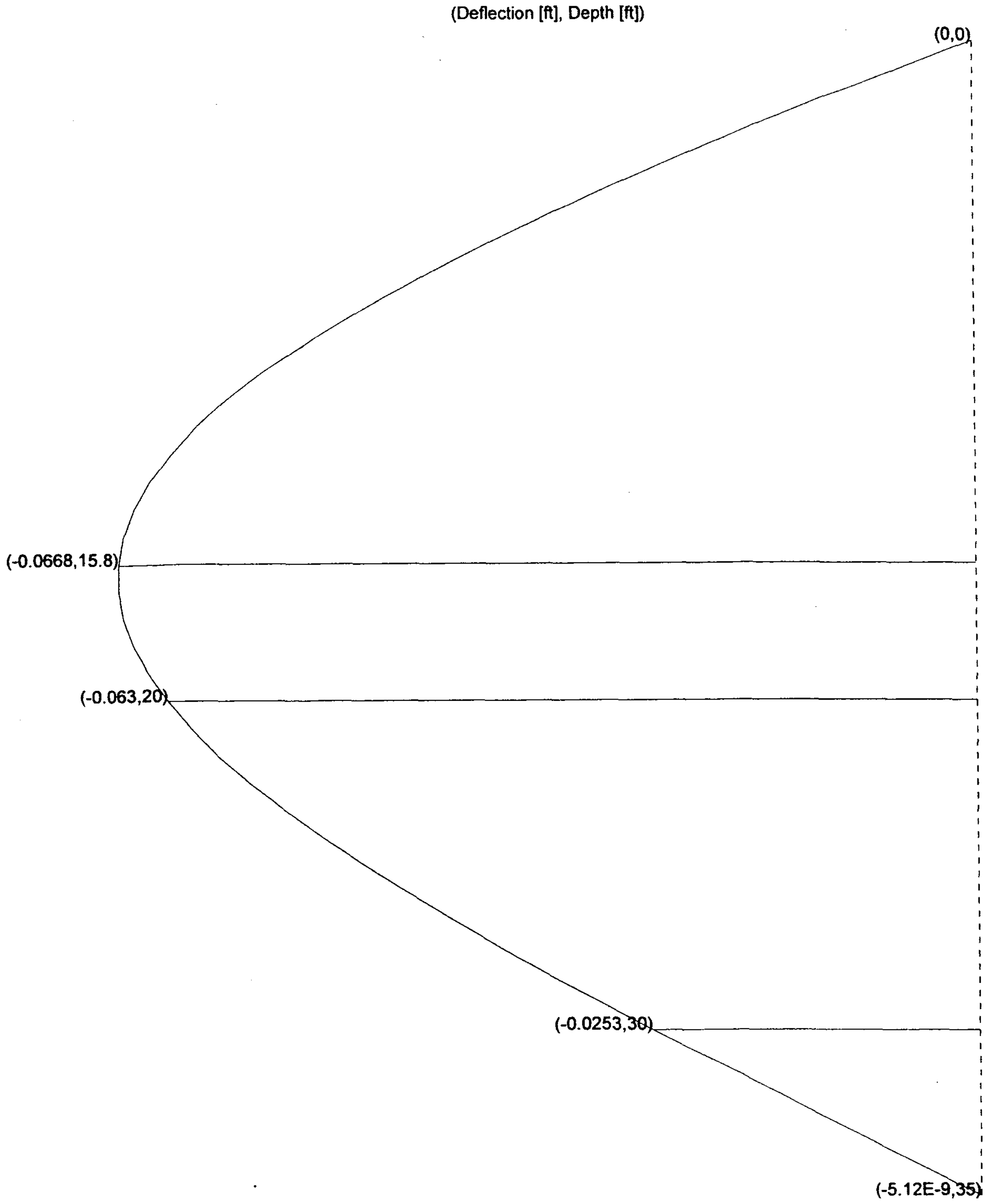
DEFLECTION DIAGRAM

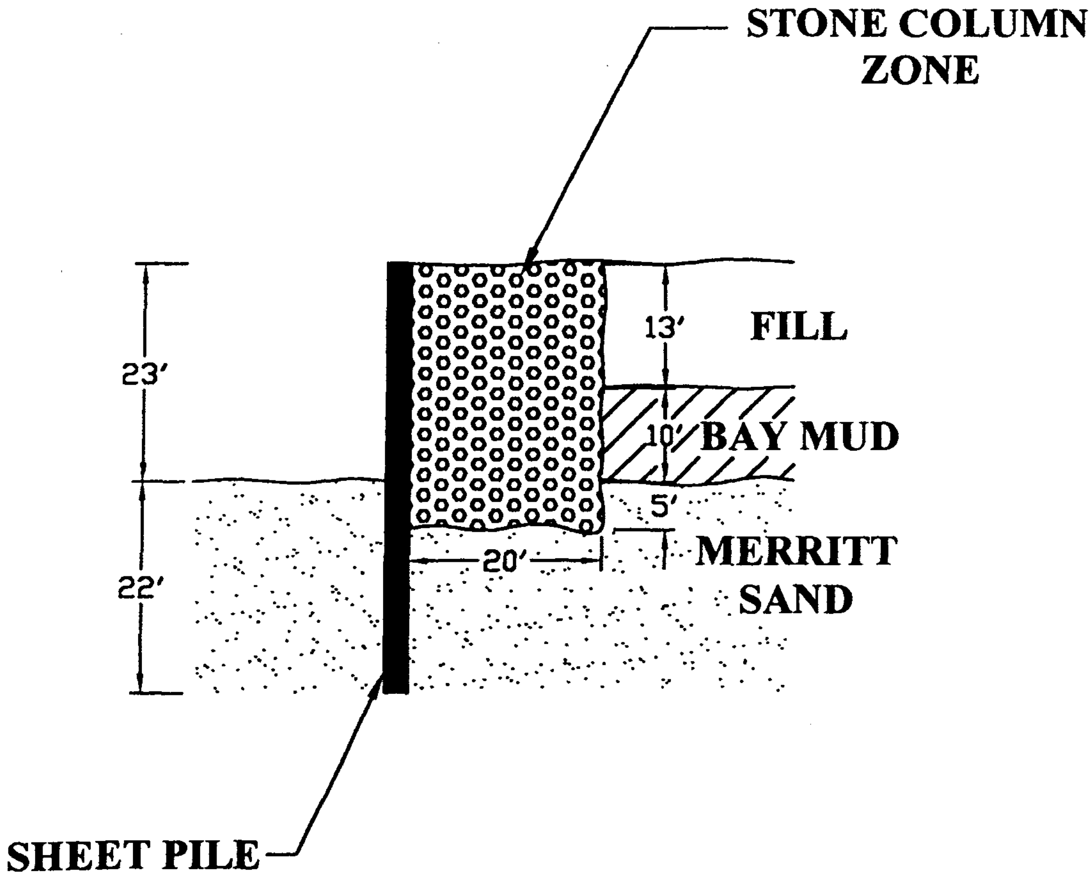


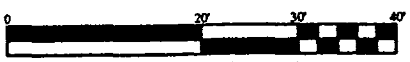
SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\FSeismic_sc_a.spc
Comment:	Section F-F' Avg'd Pre- and Post- EQ Soil 20 ft Stone Column Zone Anchored

DEFLECTION DIAGRAM







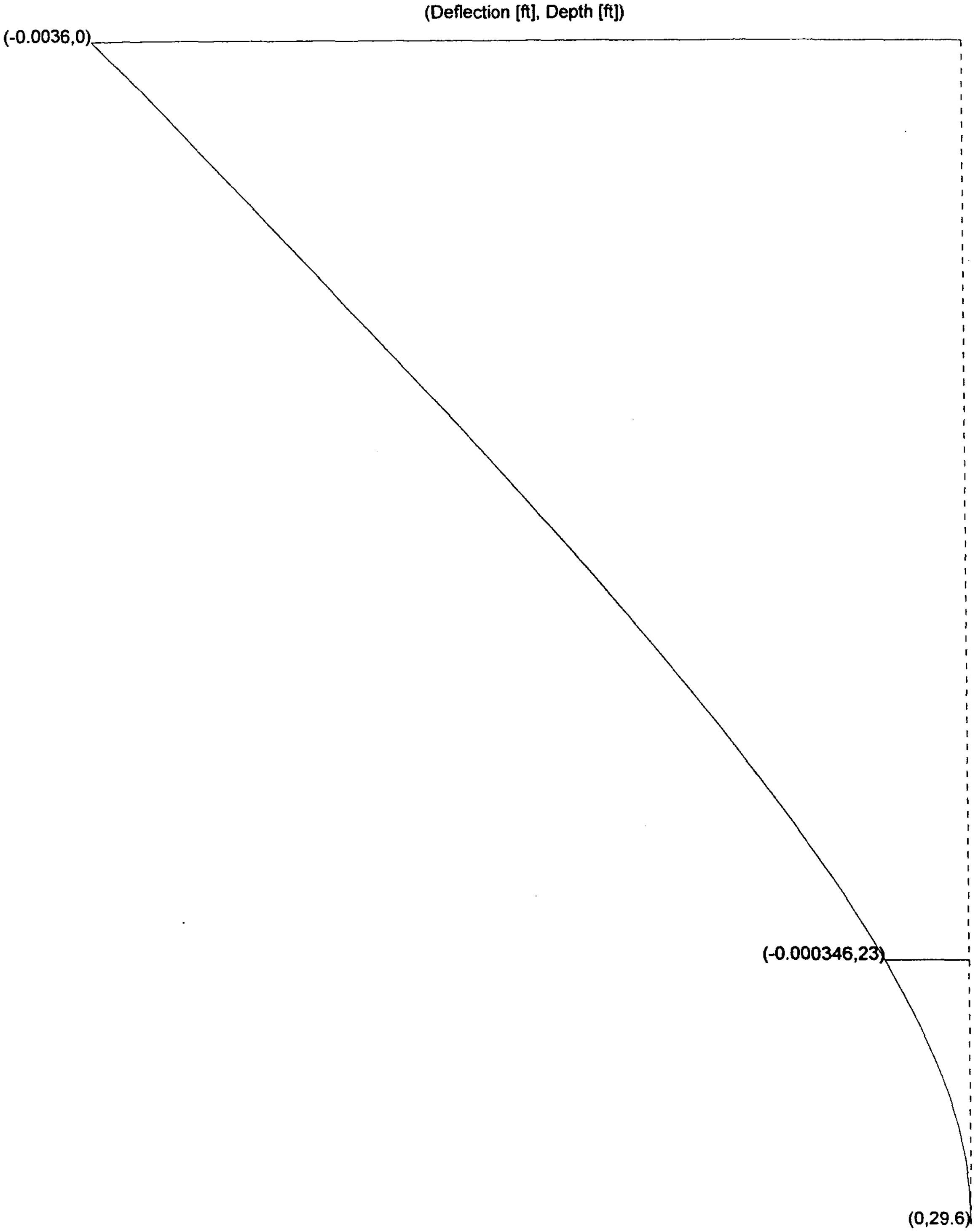
SCALE: 1" = 20'

SECTION G-G' - STONE COLUMN AND SHEET PILE -CANTILIVER-

SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\GStatic_sc.spc
Comment:	Section G-G' Post-EQ Soil Properties 20 ft Stone Column Zone Cantilevered

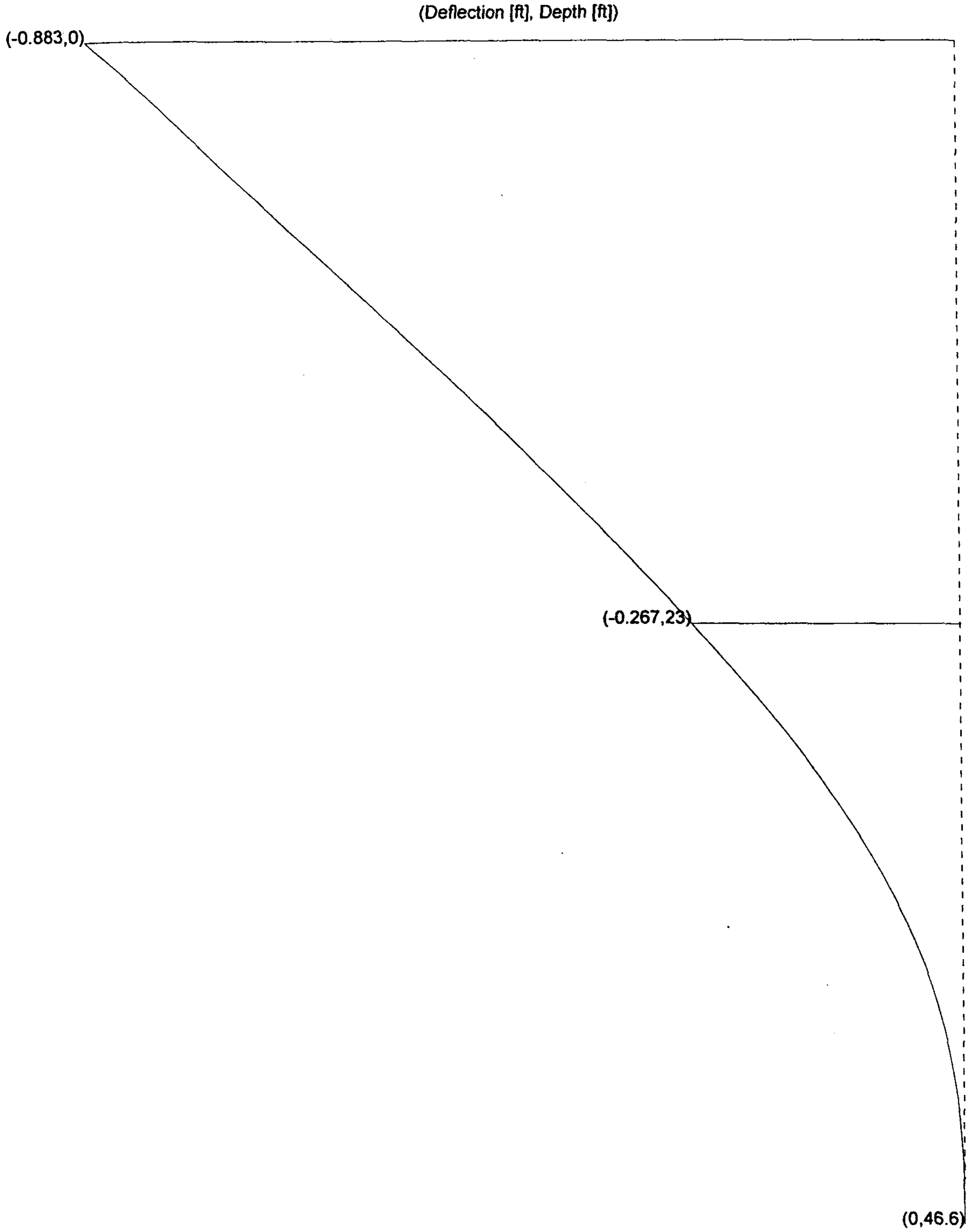
DEFLECTION DIAGRAM

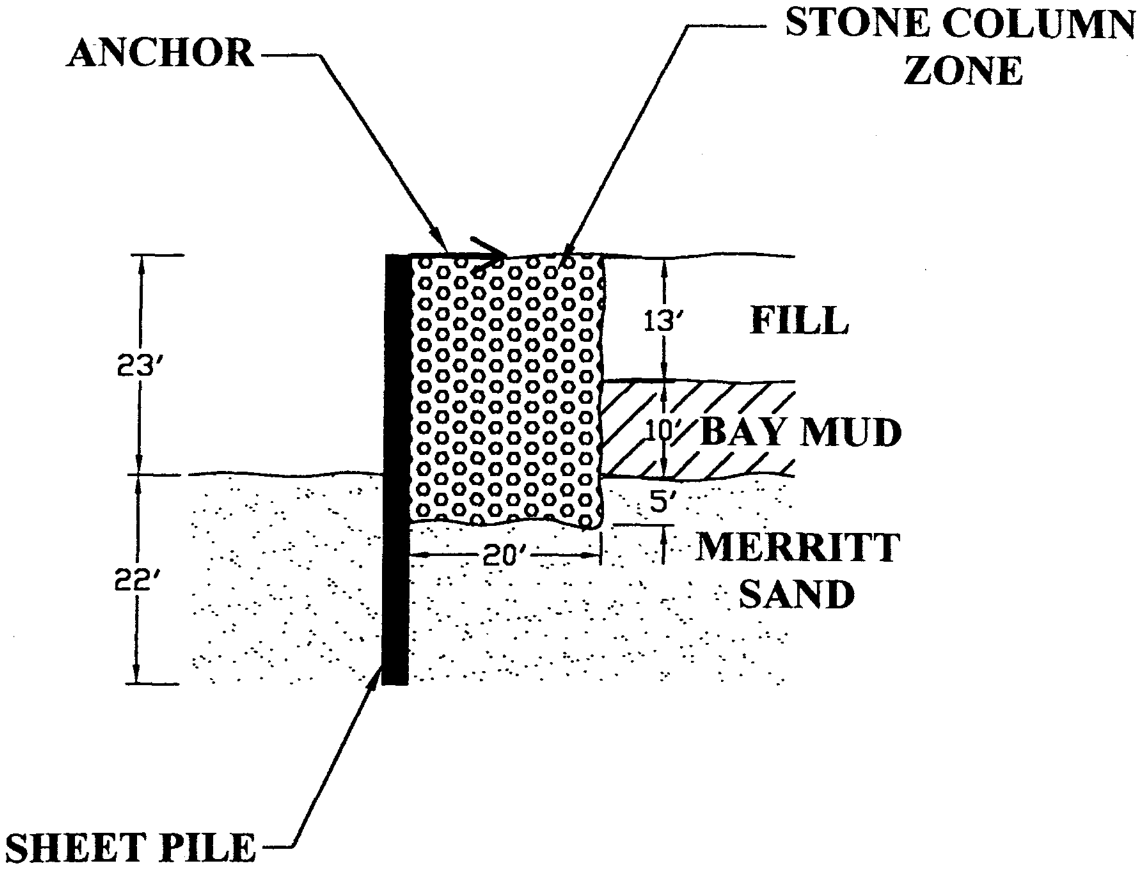


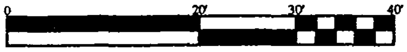
SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\GSeismic_sc.spc
Comment:	Section G-G' Avg'd Pre- and Post-EQ Soil 20 ft Stone Column Zone Cantilevered

DEFLECTION DIAGRAM







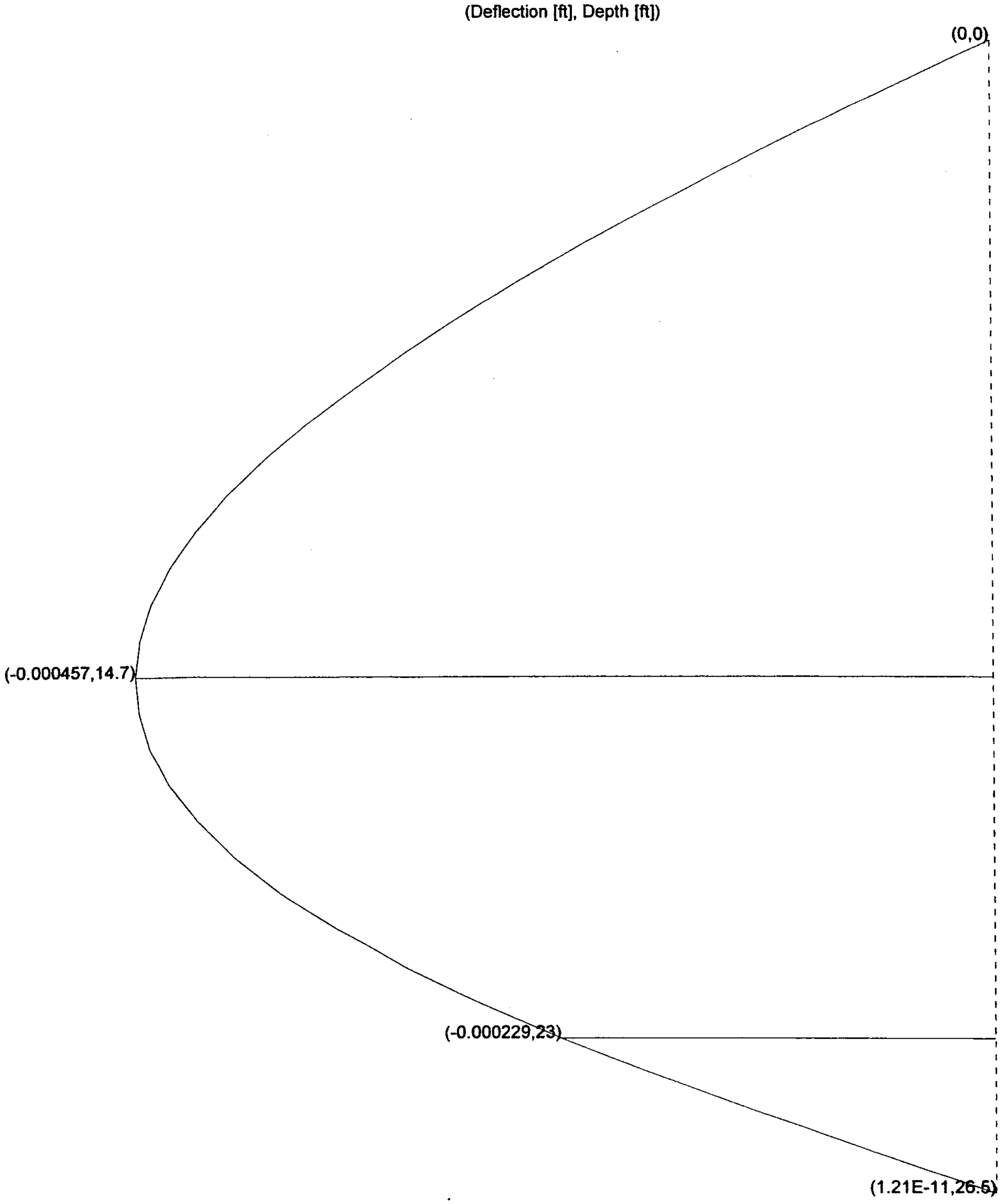
SCALE: 1" = 20'

FIGURE 20
SECTION G-G' - STONE COLUMN AND SHEET PILE
-ANCHORED-

SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\GStatic_sc_a.spc
Comment:	Section G-G' Post-EQ Soil Properties 20 ft Stone Column Zone Anchored

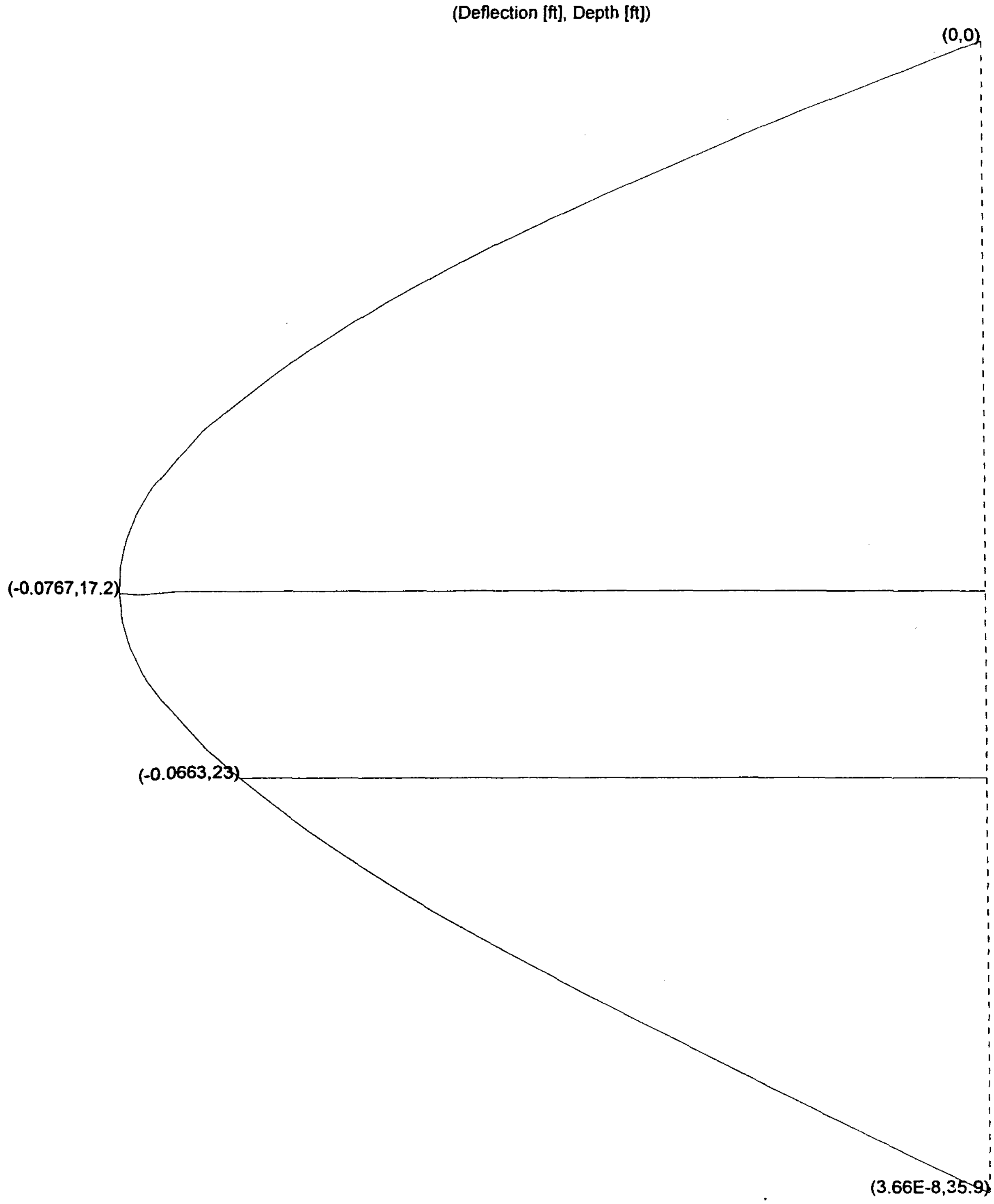
DEFLECTION DIAGRAM



SHEET PILE DESIGN
ACCORDING TO BLUM-METHOD

Date:	6/20/02
User-Name:	MMM
Project:	Alameda NAS
File-Name:	C:\Alameda 2002\GSeismic_sc_a.spc
Comment:	Section G-G' Avg'd Pre- and Post-EQ Soil 20 ft Stone Column Zone Anchored

DEFLECTION DIAGRAM

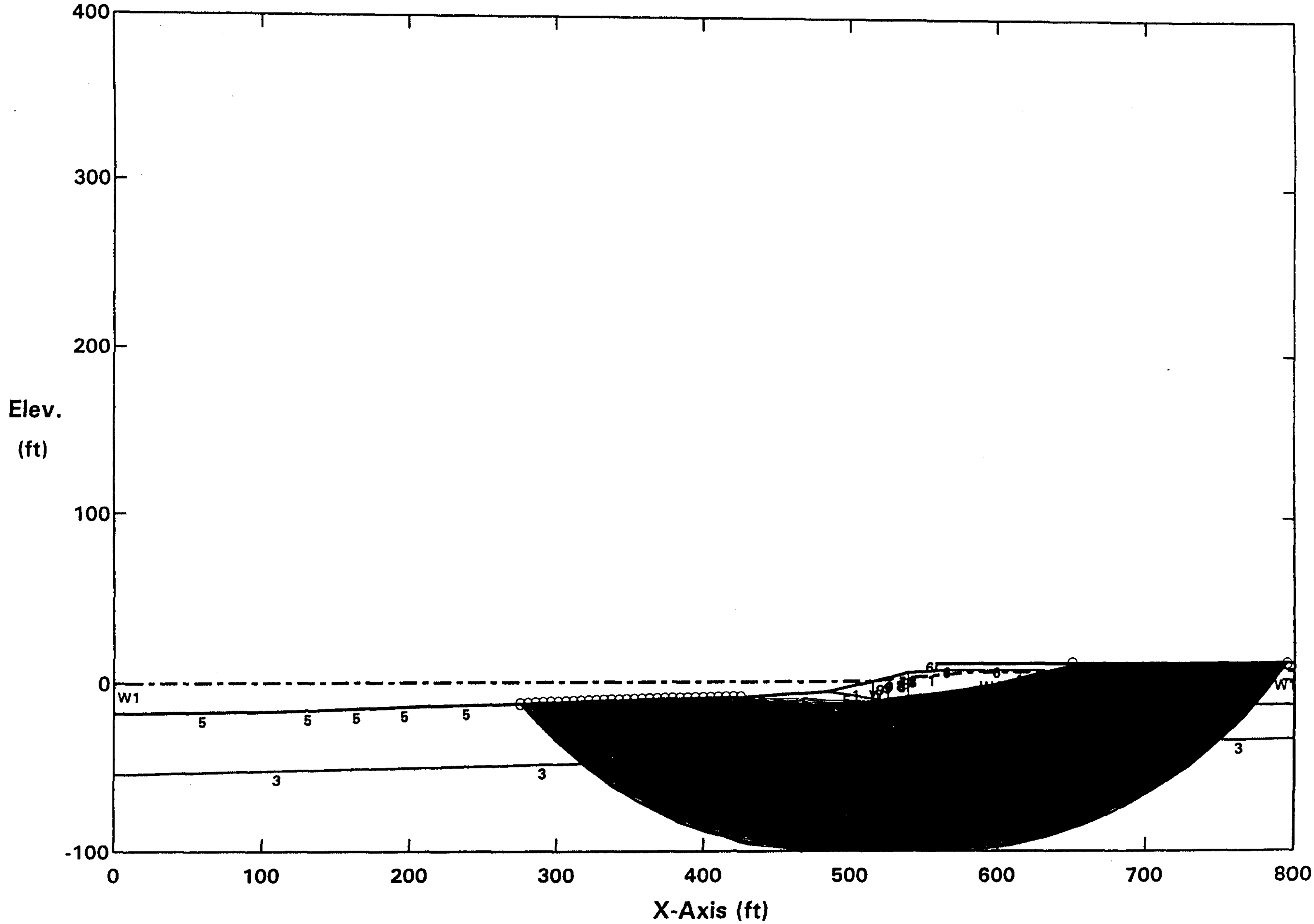


APPENDIX A5

ALTERNATIVE 5 – SOIL CEMENT GRAVITY WALL AND STONE COLUMNS

A-NAS - Section D-D', Comb., Static Long-Term Bishop Circular Search

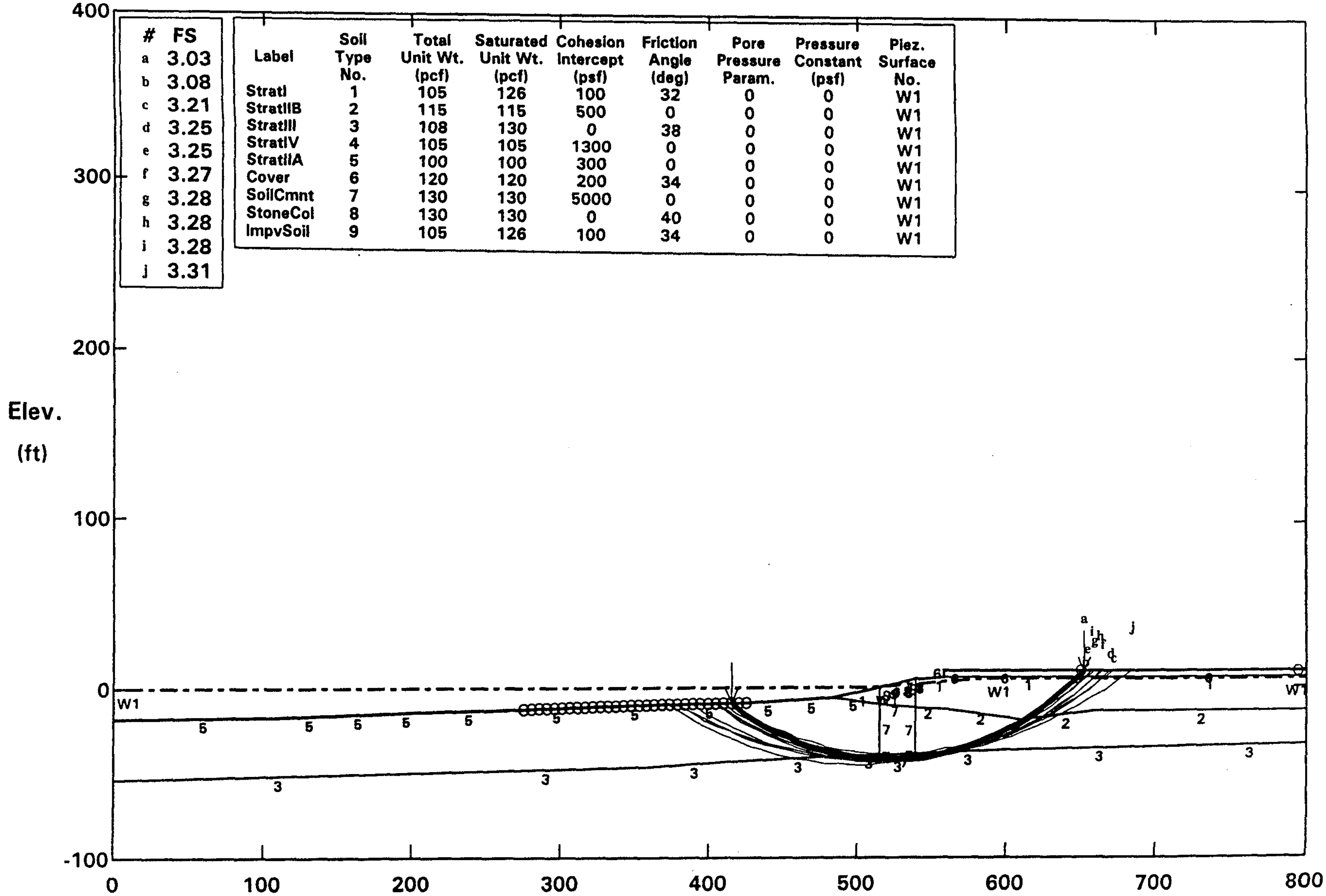
All surfaces evaluated. C:DDCMBBS1.PLT By: P.T. 07-01-02 6:41am



Factors Of Safety Calculated By The Modified Bishop Method

A-NAS - Section D-D', Comb., Static Long-Term Bishop Circular Search

Ten Most Critical. C:DDCMBBS1.PLT By: P.T. 07-01-02 6:41am



#	FS
a	3.03
b	3.08
c	3.21
d	3.25
e	3.25
f	3.27
g	3.28
h	3.28
i	3.28
j	3.31

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	100	32	0	0	W1
StratIIB	2	115	115	500	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1300	0	0	0	W1
StratIIA	5	100	100	300	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
SoilCmnt	7	130	130	5000	0	0	0	W1
StoneCol	8	130	130	0	40	0	0	W1
ImpvSoil	9	105	126	100	34	0	0	W1

PCSTABL5M FSmin = 3.03 X-Axis (ft)

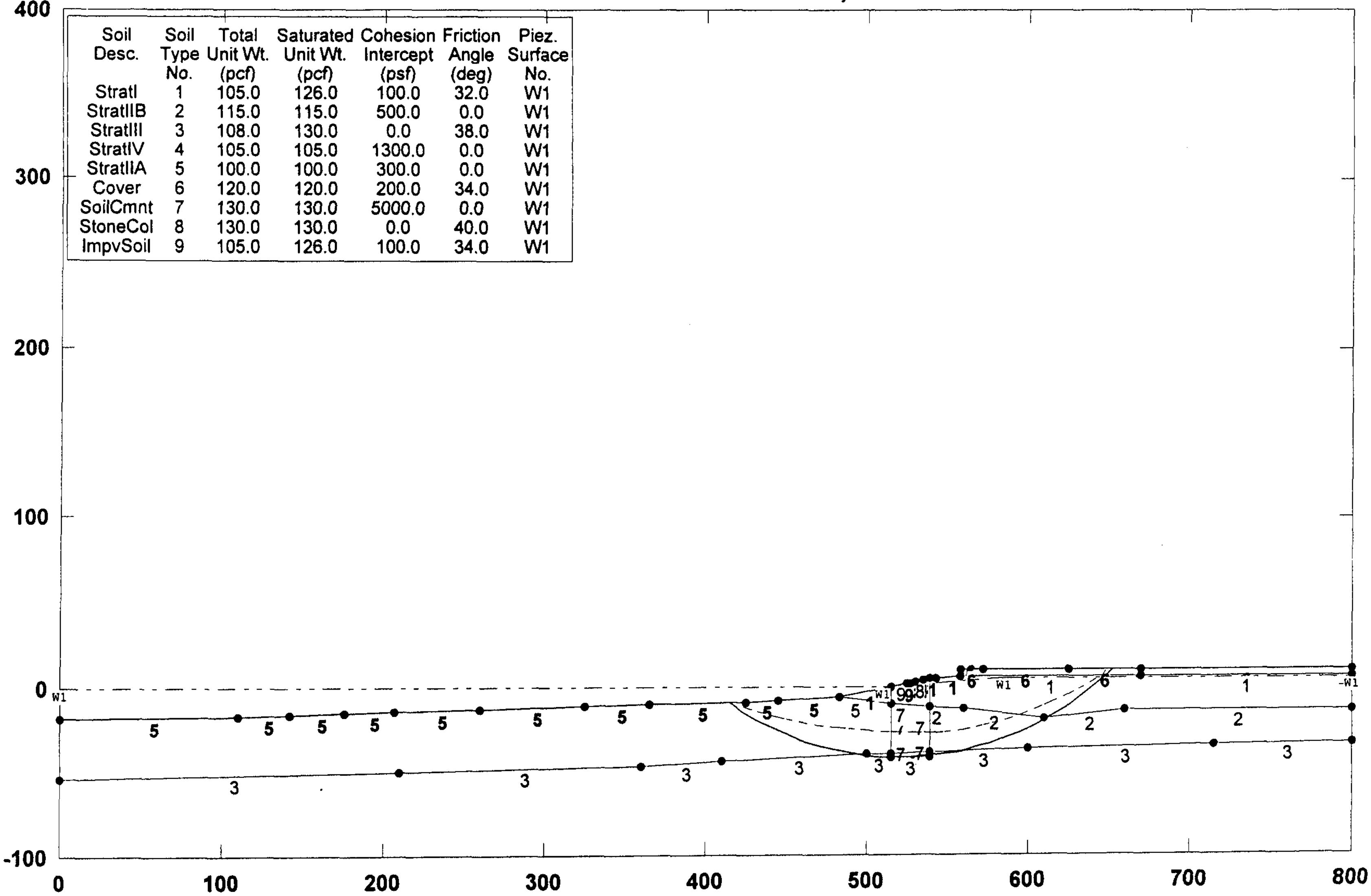
Factors Of Safety Calculated By The Modified Bishop Method

```
PROFIL       C:\GEO\STED\A-NAS\D-FINAL\DDCMBBS1.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section D-D', Comb., Static     Long-Term Bishop Circular Search
47 23
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 525. 102.1 9
525. 102.1 527. 102.5 9
527. 102.5 530. 103.1 8
530. 103.1 535. 104.1 8
535. 104.1 539. 104.6 8
539. 104.6 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 9
539. 104.6 539.1 88.4 8
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 539.1 88.4 7
539.1 88.4 560. 87. 2
539.1 88.4 539.2 61.2 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 539.2 61.2 7
539.2 61.2 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 58.5 514.9 60.45 7
539.2 61.2 539.3 58.5 7
514.8 58.5 539.3 58.5 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   SoilCmntStoneColImpvSoil
9
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1300. 0. 0. 0. 1
```

A-NAS - Section D-D', Comb., Static Long-Term Spencer Stability Anal Method

C:\GEO\STED\A-NAS\D-FINAL\DDCMBSS1.PLT Run By: PT 7/2/02 12:22PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
StratI	1	105.0	126.0	100.0	32.0	W1
StratIIB	2	115.0	115.0	500.0	0.0	W1
StratIII	3	108.0	130.0	0.0	38.0	W1
StratIV	4	105.0	105.0	1300.0	0.0	W1
StratIIA	5	100.0	100.0	300.0	0.0	W1
Cover	6	120.0	120.0	200.0	34.0	W1
SoilCmnt	7	130.0	130.0	5000.0	0.0	W1
StoneCol	8	130.0	130.0	0.0	40.0	W1
ImpvSoil	9	105.0	126.0	100.0	34.0	W1



GSTABL7 v.2 FSmin=3.05

Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

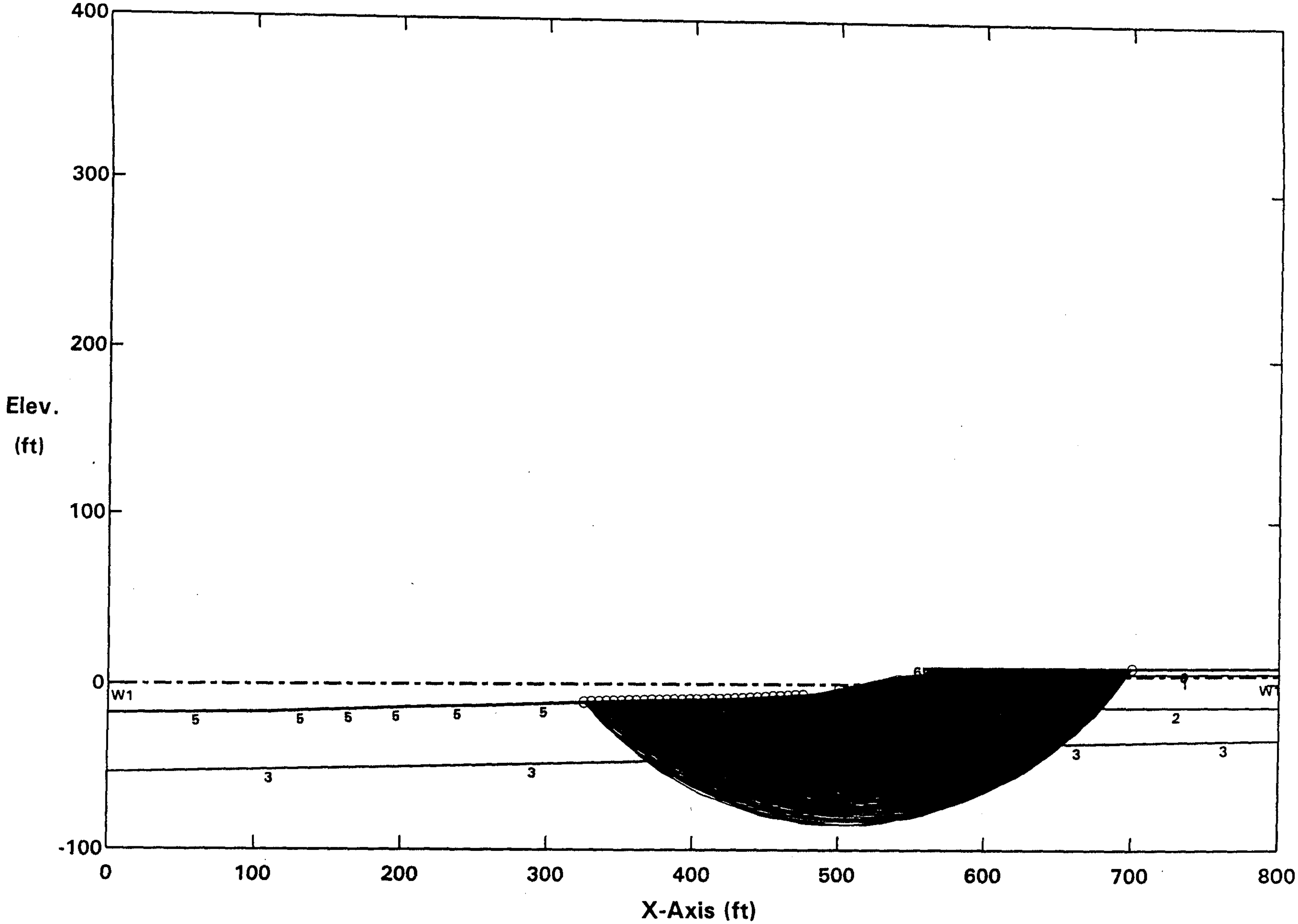
```
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 5000. 0. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
105. 126. 100. 34. 0. 0. 1
WATER
1 62.4
4
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2-Bishop circular, search
30 100
275. 425. 650. 795. 0. 10. 0. 0.
```

```
PROFIL  c:\geo\sted\a-nas\d-final\ddcmbss1.in Version G7v.2 [GSTABL72.EXE] /O(0,
-100)
e
A-NAS - Section D-D', Comb., Static     Long-Term Spencer Stability  Anal Method
47 23
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 525. 102.1 9
525. 102.1 527. 102.5 9
527. 102.5 530. 103.1 8
530. 103.1 535. 104.1 8
535. 104.1 539. 104.6 8
539. 104.6 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 9
539. 104.6 539.1 88.4 8
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 539.1 88.4 7
539.1 88.4 560. 87. 2
539.1 88.4 539.2 61.2 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 539.2 61.2 7
539.2 61.2 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 58.5 514.9 60.45 7
539.2 61.2 539.3 58.5 7
514.8 58.5 539.3 58.5 3
0.
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   SoilCmntStoneColImpvSoil
9
105. 126. 100. 32. 0. 0. 1
115. 115. 500. 0. 0. 0. 1
```

```
108. 130. 0. 38. 0. 0. 1
105. 105. 1300. 0. 0. 0. 1
100. 100. 300. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 5000. 0. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
105. 126. 100. 34. 0. 0. 1
WATER
1 62.4
4 0.5
0. 100.
510. 100.
585. 105.
800. 105.
GLEMS
10.
1       Water filled tension crack (0=no,1=yes)
0       Force Distribution (0=Single slice,1=Entire failure surf)
0       Select Method (0=Spencer,1=Morgenstern-Price)
2       ki function (Spencer=1 or 2, M-P=1, 2, 3, 4, or 5=user)
1.000   Lambda Coefficient (adjusts ki, 0.4 to 1.0)
0       Trial Lambda Adjustment option (0=no, 1=yes)
SURBIS
27
414.66 90.83
423.07 85.42
431.75 80.46
440.69 75.97
449.85 71.96
459.2 68.43
468.73 65.4
478.41 62.88
488.21 60.87
498.09 59.38
508.05 58.41
518.04 57.97
528.04 58.06
538.02 58.67
547.95 59.81
557.82 61.46
567.58 63.64
577.21 66.33
586.69 69.52
595.98 73.21
605.07 77.38
613.93 82.02
622.53 87.12
630.84 92.68
638.86 98.66
646.55 105.05
652. 110.1
EXECUT
```

A-NAS - Section D-D', Comb. Post-EQ Static Bishop Circular Search

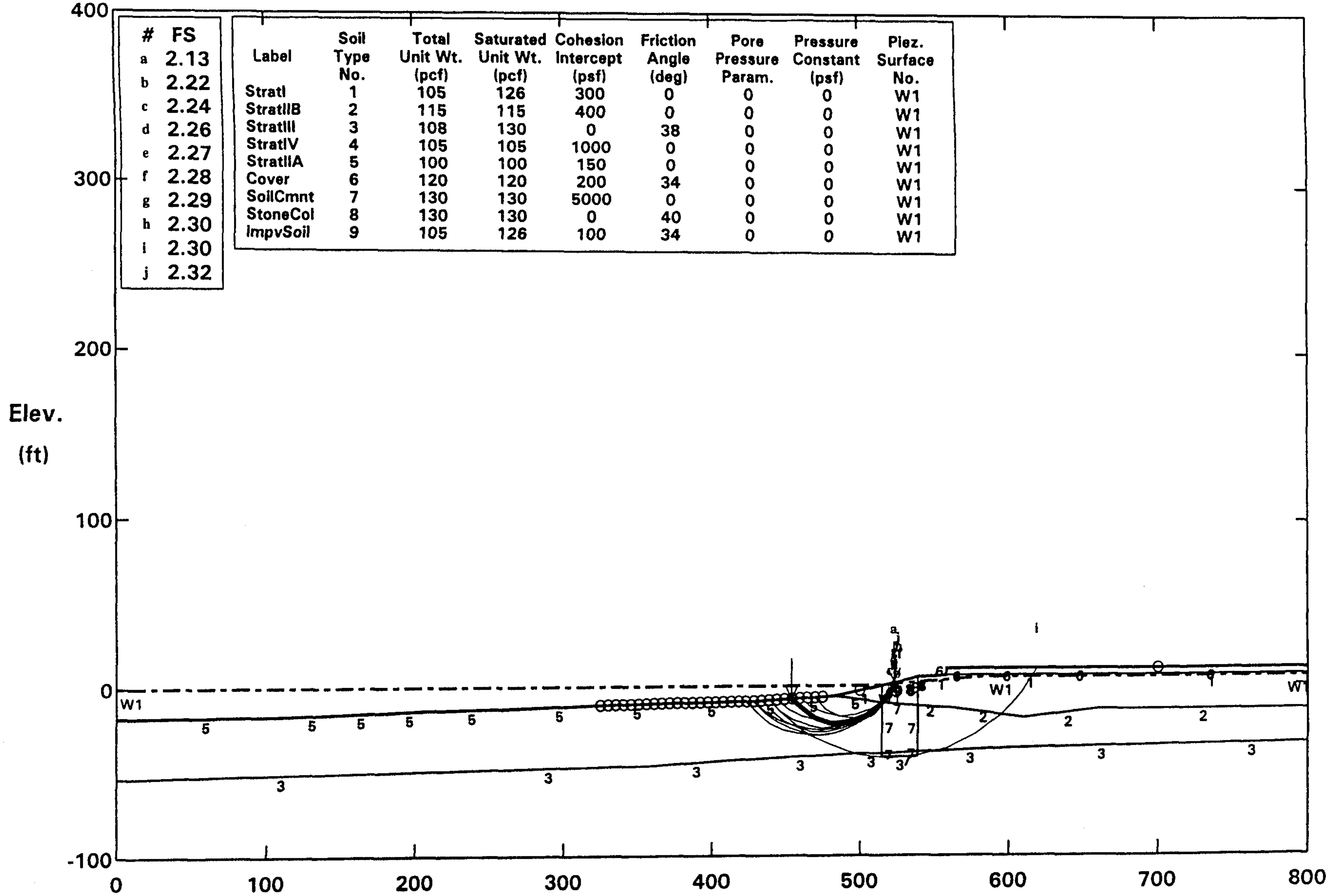
All surfaces evaluated. C:DDCMBBSE.PLT By: P.T. 07-01-02 4:13pm



Factors Of Safety Calculated By The Modified Bishop Method

A-NAS - Section D-D', Comb. Post-EQ Static Bishop Circular Search

Ten Most Critical. C:DDCMBBSE.PLT By: P.T. 07-01-02 4:13pm



#	FS
a	2.13
b	2.22
c	2.24
d	2.26
e	2.27
f	2.28
g	2.29
h	2.30
i	2.30
j	2.32

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	300	0	0	0	W1
StratIIB	2	115	115	400	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1000	0	0	0	W1
StratIIA	5	100	100	150	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
SoilCmnt	7	130	130	5000	0	0	0	W1
StoneCol	8	130	130	0	40	0	0	W1
ImpvSoil	9	105	126	100	34	0	0	W1

PCSTABL5M FSmin = 2.13 X-Axis (ft)

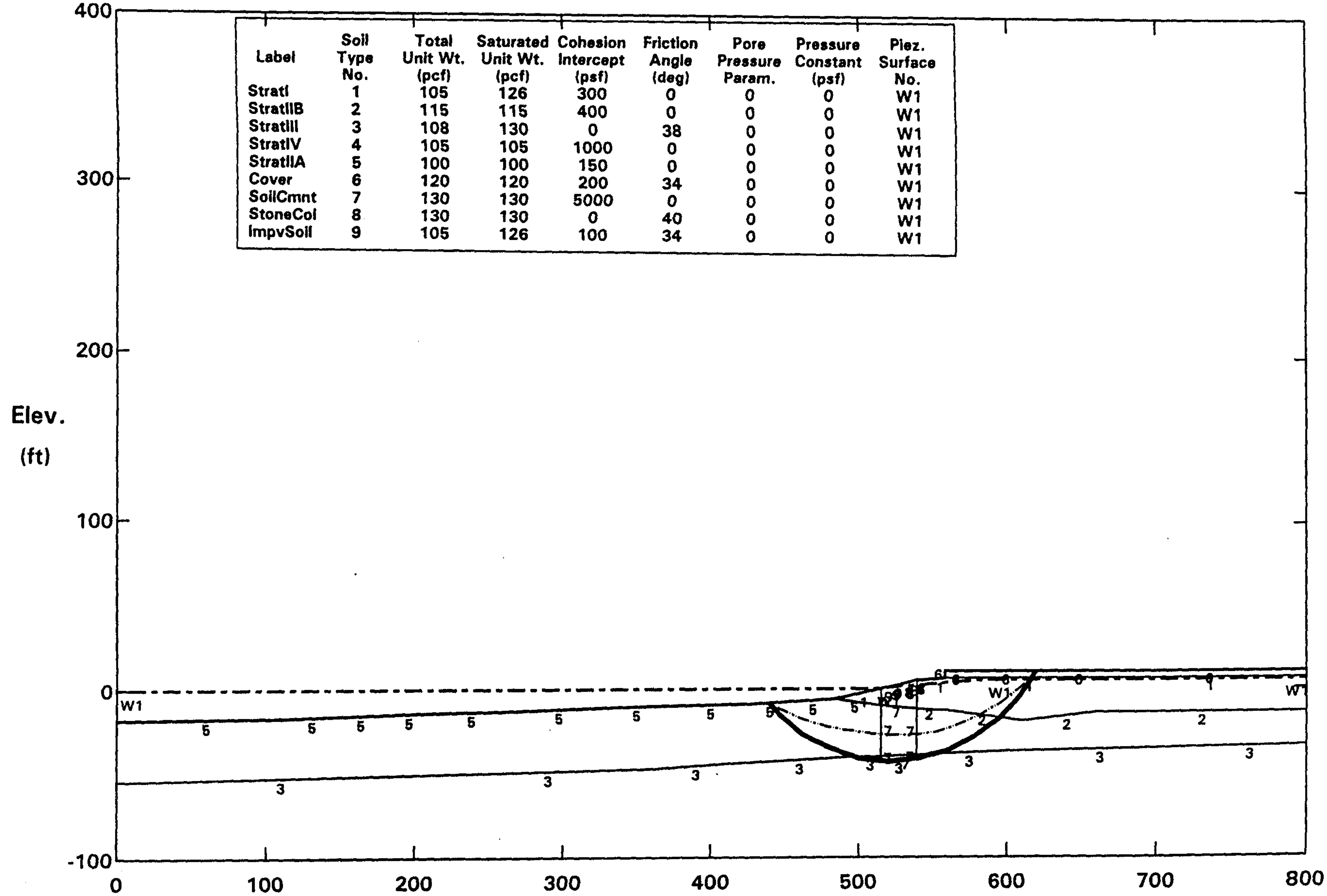
Factors Of Safety Calculated By The Modified Bishop Method

```
PROFIL       C:\GEO\STED\A-NAS\D-FINAL\DDCMBBSE.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section D-D', Comb. Post-EQ     Static Bishop Circular Search
47 23
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 525. 102.1 9
525. 102.1 527. 102.5 9
527. 102.5 530. 103.1 8
530. 103.1 535. 104.1 8
535. 104.1 539. 104.6 8
539. 104.6 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 9
539. 104.6 539.1 88.4 8
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 539.1 88.4 7
539.1 88.4 560. 87. 2
539.1 88.4 539.2 61.2 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 539.2 61.2 7
539.2 61.2 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 58.5 514.9 60.45 7
539.2 61.2 539.3 58.5 7
514.8 58.5 539.3 58.5 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   SoilCmntStoneColImpvSoil
9
105. 126. 300. 0. 0. 0. 1
115. 115. 400. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
```

```
100. 100. 150. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 5000. 0. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
105. 126. 100. 34. 0. 0. 1
WATER
1 62.4
4
0. 100.
510. 100.
585. 105.
800. 105.
CIRCL2-Bishop circular, search
30 100
325. 475. 500. 700. 0. 10. 0. 0.
```

A-NAS - Section D-D', Comb. Post-EQ Spencer Static Slope Stability

Surface #1-DDCMBBSE.OUT. C:DDCMBSSE.PLT By: P.T. 07-01-02 4:14pm



Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
StratI	1	105	126	300	0	0	0	W1
StratIIB	2	115	115	400	0	0	0	W1
StratIII	3	108	130	0	38	0	0	W1
StratIV	4	105	105	1000	0	0	0	W1
StratIIA	5	100	100	150	0	0	0	W1
Cover	6	120	120	200	34	0	0	W1
SoilCmnt	7	130	130	5000	0	0	0	W1
StoneCol	8	130	130	0	40	0	0	W1
ImpvSoil	9	105	126	100	34	0	0	W1

PCSTABL5M FS=2.36 Theta=5.77 X-Axis (ft)

Factors Of Safety Calculated By Spencer`s Method of Slices

```
PROFIL       C:\GEO\STED\A-NAS\D-FINAL\DDCMBSSE.IN  PCSTABL Version 5M /O(0. ,-
100. )
A-NAS - Section D-D', Comb. Post-EQ     Spencer Static Slope Stability
47 23
0. 82.3 110. 83. 5
110. 83. 142. 84. 5
142. 84. 176. 85. 5
176. 85. 207. 86. 5
207. 86. 260. 87. 5
260. 87. 325. 89. 5
325. 89. 365. 90. 5
365. 90. 425. 91. 5
425. 91. 445. 92. 5
445. 92. 483. 94. 5
483. 94. 515.1 100.2 1
515.1 100.2 525. 102.1 9
525. 102.1 527. 102.5 9
527. 102.5 530. 103.1 8
530. 103.1 535. 104.1 8
535. 104.1 539. 104.6 8
539. 104.6 543. 105.1 1
543. 105.1 558. 106.1 1
558. 106.1 558.1 110.1 6
558.1 110.1 572. 110.1 6
572. 110.1 625. 110.1 6
625. 110.1 670. 110.1 6
670. 110.1 800. 109.6 6
515. 90. 515.1 100.2 9
539. 104.6 539.1 88.4 8
558. 106.1 670. 106.1 1
670. 106.1 800. 105.6 1
483. 94. 515. 90. 5
514.9 60.45 515. 90. 7
515. 90. 539.1 88.4 7
539.1 88.4 560. 87. 2
539.1 88.4 539.2 61.2 7
560. 87. 610. 81.5 2
610. 81.5 660. 86. 2
660. 86. 800. 86. 2
0. 46. 210. 50. 3
210. 50. 360. 53. 3
360. 53. 410. 56. 3
410. 56. 500. 60. 3
500. 60. 514.9 60.45 3
514.9 60.45 539.2 61.2 7
539.2 61.2 600. 63. 3
600. 63. 715. 65. 3
715. 65. 800. 66. 3
514.8 58.5 514.9 60.45 7
539.2 61.2 539.3 58.5 7
514.8 58.5 539.3 58.5 3
SOIL  StratI  StratIIBStratIIIStratIV StratIIACover   SoilCmntStoneColImpvSoil
9
105. 126. 300. 0. 0. 0. 1
115. 115. 400. 0. 0. 0. 1
108. 130. 0. 38. 0. 0. 1
105. 105. 1000. 0. 0. 0. 1
```

Continue

```
100. 100. 150. 0. 0. 0. 1
120. 120. 200. 34. 0. 0. 1
130. 130. 5000. 0. 0. 0. 1
130. 130. 0. 40. 0. 0. 1
105. 126. 100. 34. 0. 0. 1
WATER
1 62.4
4
0. 100.
510. 100.
585. 105.
800. 105.
SPENCR
10.
SURBIS #1-DDCMBBSE.OUT
22
438.79 91.69
446.15 84.92
454.07 78.8
462.47 73.38
471.31 68.7
480.51 64.79
490.01 61.68
499.75 59.39
509.64 57.94
519.62 57.35
529.62 57.6
539.56 58.71
549.37 60.66
558.97 63.44
568.31 67.04
577.3 71.41
585.88 76.54
594. 82.38
601.59 88.89
608.59 96.03
614.96 103.74
619.36 110.1
EXECUT
```